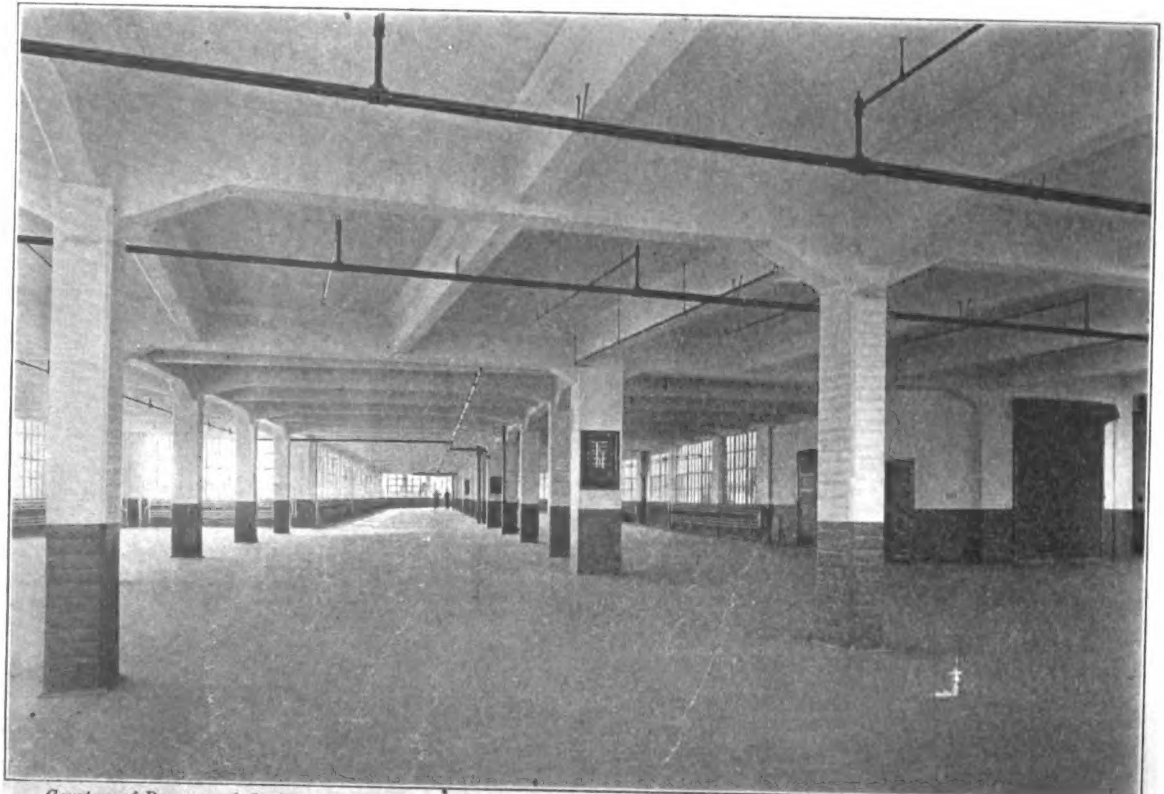
REINFORCED CONCRETE CONSTRUCTION

VOLUME II

RETAINING WALLS AND BUILDINGS



Courtesy of Densmore & Le Clear. Frontispiece.

Typical Interior of a Concrete Building.

ENGINEERING EDUCATION SERIES

REINFORCED CONCRETE CONSTRUCTION

VOLUME II. RETAINING WALLS AND BUILDINGS

PREPARED IN THE
EXTENSION DIVISION OF
THE UNIVERSITY OF WISCONSIN

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STANDARD NOTATION

USED THROUGHOUT THIS VOLUME. SEE VOL. I, OR SECTION I OF THE APPENDIX.

PREFACE

This volume on reinforced-concrete construction treats of the design and the construction of retaining walls and buildings. It has been prepared to meet the needs of students having varying qualifications, and on this account is more comprehensive in some respects than the ordinary text-book dealing with these subjects. The designing work is given in detail and an effort has been made to gain clearness by the selection and arrangement of the subject matter and by numerous drawings and photographs.

Volume I, which treats of the fundamental principles of reinforced-concrete construction, has been adopted as a text-book in a number of technical schools and it is quite probable that the present volume will be considered for the same purpose. Since engineering courses are commonly subject to severe time limitations, it is hardly to be expected that the entire book can be covered in class-work. The experienced instructor, however, will recognize at once that many parts of the text may be omitted without seriously interfering with the continuity of the subject.

The author desires to acknowledge his indebtedness to Mr. Leslie H. Allen of the Aberthaw Construction Company for the chapters on "Estimating"; and to Mr. A. W. Ransome, Vice-President of the Ransome Concrete Machinery Company, for the chapter on "Construction Plant." These chapters should prove valuable, not only to engineering students, but to engineers in general practice. The author also wishes to express his appreciation of the excellent work of Mr. Frank C. Thiessen who made all the drawings for illustrations with the exception of those pertaining to construction plant.

The author is under obligation to Mr. A. B. MacMillan of the Aberthaw Construction Company and to Mr. H. Russell Smith of the Otis Elevator Company for critical reading of portions of the text and for many valuable suggestions. Grateful acknowledgments are due to the Universal Portland Cement Co., Turner Construction Co., Atlas Portland Cement Co., Ferro-Concrete Construction Co., Concrete Engineering Co., C. A. P.

Turner Construction Co., Aberthaw Construction Co., Otis Elevator Co., Unit Construction Co., Raymond Concrete Pile Co., Simplex Foundation Co., Densmore and LeClear, Mr. Arthur Peabody, Mr. Benjamin Fox, Mr. Allen Brett, Mr. Harry F. Porter, and Mr. A. C. Chenoweth for the use of plans and photographs.

G. A. H.

THE UNIVERSITY OF WISCONSIN,
MADISON, WISCONSIN,
October 1, 1913.

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REINFORCED CONCRETE CONSTRUCTION

PART I

RETAINING WALLS

A retaining wall is a wall of masonry built to sustain the lateral pressure of earth or of other material possessing more or less frictional stability.

A reinforced-concrete retaining wall is nearly always more economical than a retaining wall of gravity section, either of stone masonry or plain concrete. In stone masonry and plain concrete walls the section must be made heavy enough so that the weight of the structure will prevent overturning, while in reinforced-concrete walls the weight of a considerable part of the sustained material is utilized and, in addition, the sections may be designed to more nearly develop the full strength of the concrete.

CHAPTER I

THEORY OF STABILITY

1. Earth Pressure.—To determine the effect of the thrust of a bank of earth against a wall, it is necessary to know (1) the magnitude of the pressure, (2) its point of application, and (3) its line of action. We have little exact knowledge, however, concerning these three elements of the thrust.

In Fig. 1, let *AB* represent the back of a retaining wall, and *AC* the surface of the ground. The earth has a tendency to break away and come down some line, as *CB*, thus producing pressure on the wall. The weight of the earth tends to cause

this breaking away while the resisting forces are the friction and the cohesion along the line *BC*, and the resistance of the wall. Unfortunately there is a lack of adequate experimental data concerning the constants of weight, friction, and cohesion of any particular ground, and this lack of information on the subject has led to some extent to the use of formulas based almost wholly upon purely theoretical considerations.

Earth pressure theories neglect cohesion of the filling and a number of experiments have been made to check up the theoretical deductions, using material approaching as nearly as possible to the ideal condition of a granular mass. Clean, dry sand has been used extensively in such experiments inasmuch as it has

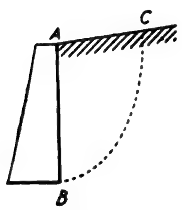


FIG. 1.

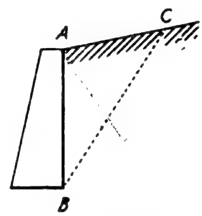


FIG. 2.

little cohesion and is about as good as any material for the purpose. However, the experiments that have been made are of little value, due principally to the difficulty of devising a measuring apparatus that will give the true pressures. Some experiments have also been made on cohesive material, but the problem here is even a much more difficult one than the determination of the pressures of granular materials and the results do not help appreciably in the determination of earth pressures for the conditions found in practice. Most of the tests on retaining walls have been made with material of limited extent, so that the results are not comparable.

2. Earth Pressure Theories.—All earth pressure theories assume (1) that the surface of rupture, *BC*, Fig. 1, is a plane, (2) that the point of application of the resultant pressure is at one-third of the height of the wall from the bottom, and (3) that the resultant pressure makes a definite angle with the horizontal. Each of these three assumptions has been seriously questioned.

Assuming that the surface of rupture is a plane, is equivalent to assuming that the soil is devoid of cohesion, and that it is homogeneous and non-compressible. Consider for a moment a material which is so thoroughly pulverized that all cohesion between its particles is destroyed. If this material were poured vertically upon a horizontal surface, the surface slope would make an angle with the horizontal whose tangent would be equal to the coefficient of friction—called the *angle of repose* of the material. The particles on such a slope would be held in equilibrium by the force of gravity and by friction. Now consider this material placed behind a retaining wall, as in Fig. 2. If *BC* is the plane of rupture, the prism *ABC* is what is called the *sliding wedge* and the resultant pressure on *BC* makes an angle with the normal to *BC* equal to the coefficient of friction of the material. This angle is called the angle of internal friction and experiments show that this angle differs materially from the angle of surface slope. The resistance of particles moving on an exposed slope is probably rolling friction rather than sliding, while the resistance involved in the lateral pressure of earth is sliding friction. There seems to be no constant relation between the values obtained from these two kinds of friction. The angle of internal friction seems to depend upon the size of the particles and to increase with the pressure and the moisture, but additional experiments are needed to determine the law of its variation.

In a granular mass of earth we have seen that there is a limit to the angle which the direction of the pressure on a plane can make with the normal to that plane. But suppose the earth is not a granular mass. Then its cohesion (meaning the force uniting the particles of the earth, whether that force be adhesion or true cohesion) acts like the cohesion of a solid body and the angle of internal friction may be 90°, or even greater than 90°; that is, the earth may be cut to stand vertically, or even overhanging. Cohesion, however, is influenced greatly by the effect of moisture and the vibration of moving loads and is partially, if not entirely, destroyed when earth is removed from its original location. Thus, earth pressure theories, based on a perfectly dry granular mass, tend to give maximum conditions and this is an argument advanced for their use.

In a liquid like water, the pressure at any point is exerted in all directions with equal intensity, acts normal to any given

surface, and depends only upon the depth below the surface, or the *head* as it is called. Let h be the head and w the weight of a cubic unit of the liquid. Then at the depth h, one horizontal square unit sustains a pressure equal to the weight of a column of liquid whose height is h, and whose cross-section is one square unit, or wh. Now, since fluid pressure is exerted in all directions with equal intensity, the unit pressure on a wall which prevents the flow of a liquid will increase directly as the depth, and the unit pressures may be graphically represented by arrows which form a triangle, as in Fig. 3. If a force P equal to the total pressure be applied at the center of gravity of the traingle, or $\frac{1}{3}h$ above the base of wall, it will have an effect identical to that

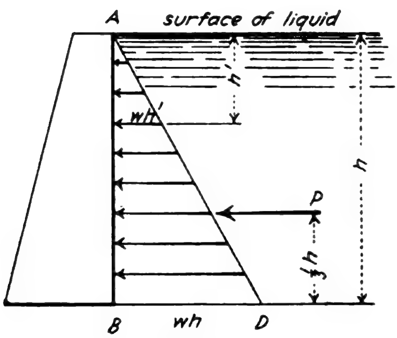


FIG. 3.

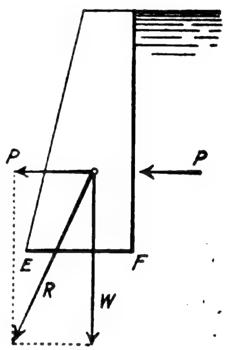


FIG. 4.

of all the unit-pressures on the wall. Thus, the total pressure on a wall due to a liquid may be obtained by finding the area of triangle ABD, Fig. 3, or

$$P = wh.\tfrac{1}{2}h = \tfrac{1}{2}wh^2$$

The fluid pressure P and the weight of wall W, Fig. 4, cause a force R (which is the resultant of P and W) to act upon the base of wall. Now since in all earth-pressure theories, the amount of the lateral pressure varies as h^2—that is, as the square of the vertical height of the wall—it is always assumed that earth pressure follows the law of liquid pressure and that therefore the point of application is $\frac{1}{3}h$ from the bottom of the wall. The argument advanced against this assumption is that it has not been proved beyond question that the lateral pressure of earth

varies as the square of the height, and hence that it cannot be concluded that the point of application is certainly at one-third the height; also, that the prism of earth between the plane of rupture and the back of the wall has been assumed, in deducing the formula for the amount of pressure, to act as a solid wedge sliding on the plane of rupture and there is thus no reason for claiming that the pressure will be applied at one-third the height any more than at the center of the height. Since earth is neither a liquid nor a solid, it is quite probable that the true point of application of the resultant pressure is somewhere between these two extremes, and experiments tend to show some basis for this belief.

The results by the different earth-pressure theories differ considerably and this difference is due principally to the different assumptions as to the direction of the resultant earth pressure. For a level earth surface and a vertical back to the retaining wall (the common case) the ordinary theories do not differ greatly and it is generally conceded that all give results that are considerably on the safe side. But if the theories are applied to other than common cases, inconsistencies crop out first in one theory and then in another, which shows that the underlying assumptions are inconsistent. In fact, most of the theories are at variance with experiments and experience, and they all contain logical contradictions.

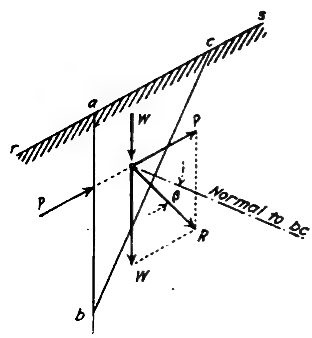


FIG. 5.

A working knowledge of Rankine's Theory will be given and, since this theory considers both active and passive earth pressures, an explanation of what is meant by these terms may serve to give the student a better grasp of the subject. With a fluid like water, the resultant pressure on any plane is normal to that plane due to a lack of friction between the particles. With a mass of granular material, on the other hand, friction does exist between the particles and the resultant pressure may make an angle with the normal less than or equal to the angle of internal friction, and hence its magnitude consistent with equilibrium may vary considerably. In Fig. 5 let *rs* represent the earth surface, and let the mass of earth be divided by the

plane *ab*; also consider the left-hand portion removed. Now, if the total pressure on *ab* before the earth mass was divided be designated by *P*, this force if applied to *ab* at the center of pressure will clearly hold the earth to the right of *ab* in equilibrium. Let *W* equal the weight of a prism such as *abc*. It is evident that the pressure on the plane *bc* will be *R*, the resultant of *P* and *W*, and the angle β which it makes with the normal to *bc* must be less than the angle of internal friction if the earth does not slide on *bc*. This must be true for any plane *bc* passing through *b*, and there will be one of these planes on which the tendency to slide will be the greatest—that is, on the plane of rupture—which will require the largest value of *P* in order that *B* may just equal the angle of internal friction. This is the minimum value of *P* which will hold the earth in equilibrium, and it is the pressure which any retaining wall with back *ab* must be capable of resisting. This pressure is called the *active* earth pressure, because the earth is active in pressing against the wall, and the wall simply prevents the earth from sliding down. If now *P* is increased in any way, as by letting an arch abut against the wall, the angle β will be reduced and, if the magnitude of *P* is increased sufficiently, this angle will become negative. When the value of *P* is such that the resultant pressure on some plane *bc* makes an angle above the normal equal to the angle of internal friction, the earth is on the point of sliding upward. This will occur on the plane having the greatest tendency to slide; that is, it will occur on the plane requiring the smallest value *P* in order to cause the resultant to make the limiting angle with the normal to *bc*. This value of *P* is called the *passive* earth pressure because there is an active force existing to force the earth up, instead of passively resisting its tendency to slide down. In the case of a retaining wall we usually need to deal only with active earth pressure, but in some cases passive earth pressure must be considered.

3. Rankine's Earth-pressure Theory.—Rankine's Theory is based upon the principles governing the distribution of stress in a rigid body. It is assumed that the filling consists of an incompressible, homogeneous, granular mass, without cohesion; that the mass is of indefinite extent; that the upper surface is a plane; and that on no plane passing through a given point does the obliquity of stress exceed the angle of internal friction, while on one plane it just equals it.

The general formulas evolved by Mr. Rankine from the assumptions given above apply to both gravity walls and reinforced walls.

Let P = resultant earth pressure in pounds on a vertical surface for a length of wall equal to 1 ft., Fig. 6.
h = total height of surface in feet.
w = weight of earth per cubic foot.
θ = angle of inclination of earth behind the wall.
ϕ = angle of internal friction of the earth.

Then

$$P=\tfrac{1}{2}\,wh^2\cos\theta\,\frac{\cos\theta\mp\sqrt{\cos^2\theta-\cos^2\phi}}{\cos\theta\pm\sqrt{\cos^2\theta-\cos^2\phi}}$$

In this equation the negative sign in the numerator should be used with the positive sign in the denominator to give the active pressure. For passive pressure the signs should be reversed.

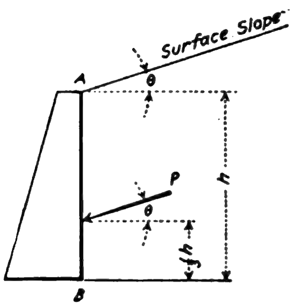


FIG. 6.

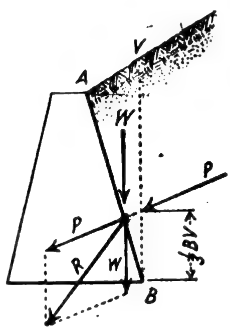


FIG. 7.

The pressure is assumed to act parallel to the slope of the surface of the earth, and for walls with no load upon the filling it acts at one-third the height of the wall from the base.

When the earth surface is horizontal, we have

$$P=\tfrac{1}{2}\,wh^2\,\frac{1\mp\sin\phi}{1\pm\sin\phi}$$

and the pressure is horizontal.

The general expression for P which will apply to both cases is as follows:

$$P=\frac{1}{2}wh^2C$$

in which C is a coefficient.

The above formulas enable us to find the pressure on a vertical plane. If the back of the retaining wall is not vertical, as AB, Fig. 7, it is necessary to compute the pressure on the vertical plane BV passing through the back corner of the wall and combine this pressure with the weight of the prism of earth ABV (see Fig. 7).

The weight of earth and the angle of internal friction of earth to be used as filling should be known as accurately as possible when employing the above formulas. Taylor and Thompson give the following weights per cubic foot of dry material before excavation. As fills will eventually assume much the same character as earth in original excavation, the figures may be employed for either natural earth or filled material.

WEIGHTS PER CUBIC FOOT OF DRY MATERIAL BEFORE EXCAVATION

Sand	105
Gravel	135
Gravelly clay	130
Loam	90
Hard pan	130
Dry muck	40

Wet material in original excavation may weigh considerably more than dry. Gravel containing ordinary moisture weighs only about 2 per cent more than dry gravel, but wet sand if very fine may weigh as much as 10 per cent more than dry sand. If a soil is not adequately drained, it will become saturated with water and in this condition its weight may be 20 to 50 per cent greater than in a dry condition, depending upon the character of the material.

The following table gives various values of the angle of internal friction:

COEFFICIENTS OF INTERNAL FRICTION[1]

Kind of material	Tangent of angle of internal friction	Approximate corresponding Angle	Approximate corresponding Slope	Authority
Coal, shingle, ballast, etc. .	1.423	54°	0.7 to 1	Baker
Bank sand	1.423	54°	0.7 to 1	Goodrich
Riprap	1.097	48°	0.9 to 1	Goodrich
Earth	1.097	48°	0.9 to 1	Baker
Quicksand, 100-up	0.895	42°	1.1 to 1	Goodrich
Clay	0.895	42°	1.1 to 1	Baker
Quicksand, 50-100	0.750	37°	1.3 to 1	Goodrich
Earth	0.750	37°	1.3 to 1	Steel
Bank sand	0.750	37°	1.3 to 1	Wilson
Sand, 50-100	0.549	29°	1.8 to 1	Goodrich
Bank sand	0.549	29°	1.8 to 1	Goodrich
Clay	0.474	25°	2.1 to 1	Goodrich
Cinders	0.474	25°	2.1 to 1	Goodrich
Gravel, ½-in.	0.474	25°	2.1 to 1	Goodrich
Gravel, ¼-in.	0.350	19°	2.9 to 1	Goodrich
Bank sand	0.350	19°	2.9 to 1	Goodrich
Sand, 30-50	0.258	14°	3.9 to 1	Goodrich
Sand, 20-30	0.179	10°	5.6 to 1	Goodrich

Illustrative Problem.—Determine the active earth pressure on the surface AB of the wall shown in Fig. 8. Assume weight of earth = 100 lb. per cubic foot and the weight of the masonry = 150 lb. per cubic foot. Assume $\phi = 35°$

$$\cos\theta = 0.866 \qquad \cos^2\theta = 0.750$$
$$\cos\phi = 0.819 \qquad \cos^2\phi = 0.671$$
$$\sqrt{\cos^2\theta - \cos^2\phi} = 0.281$$
$$\tfrac{1}{2}wh^2\cos\theta = \tfrac{1}{2}(100)(22.31)^2(0.866) = 21{,}550$$
$$P = 21{,}550\frac{0.866-0.281}{0.866+0.281} = 11{,}000 \text{ lb.}$$
$$P = \tfrac{1}{2}wh^2\cos\theta\frac{\cos\theta \mp \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta \pm \sqrt{\cos^2\theta - \cos^2\phi}}$$

This pressure P acts on the vertical plane BV and is parallel to the earth surface. To determine the pressure on AB, the force P should be combined with the weight of the prism ABV. Since the resultant pressure on BV acts as $\frac{1}{3}$ BV above the base of wall, the resultant of this and the weight of the prism intersect at the wall surface.

The weight of prism $ABV = \frac{(100)(4)(22.31)}{2} = 4462$ lb.

[1] E. P. Goodrich, Trans. Amer. Soc. of C. E., vol. 53, p. 301.

The horizontal and vertical components of the force of 11,000 lb. are 9530 lb. and 5500 lb. respectively; hence the H and V components of the resultant pressure on AB are as indicated in Fig. 9.

When the earth behind a wall is loaded in any way—for example, when the embankment is used as a storage of material

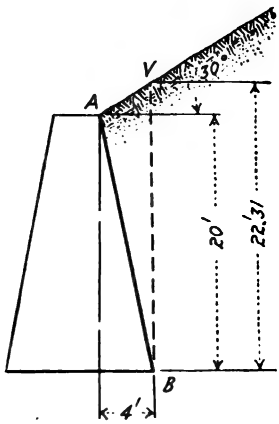


FIG. 8.

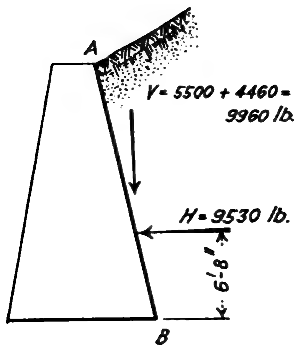


FIG. 9.

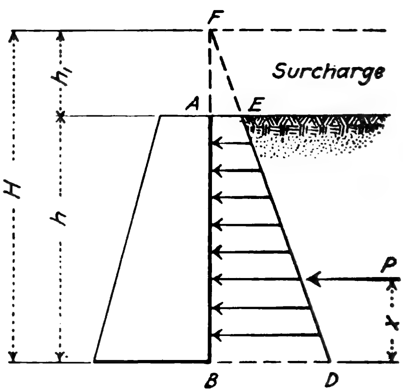


FIG. 10.

or when a railroad track runs along the wall—the additional pressure may be provided for by replacing the load by an equivalent surcharge of earth. The height of this surcharge may be determined by dividing the extra load per square foot by the

weight of a cubic foot of earth. This height is shown in Figs. 10 and 11 as h_1. Let $h+h_1=H$. Then the resultant pressure on a vertical plane for a wall with height H will be

$$P_2=\tfrac{1}{2}wH^2C$$

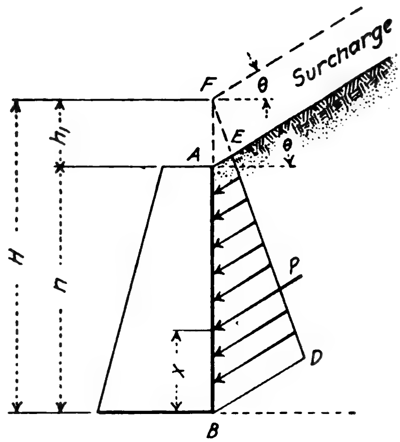


FIG. 11.

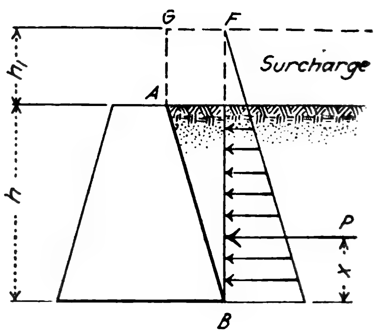


FIG. 12.

and the resultant pressure for a wall with height h_1 will be

$$P_1=\tfrac{1}{2}wh_1^2C$$

The pressure on the vertical wall AB is the difference of these, or

$$P=P_2-P_1=\tfrac{1}{2}w(H^2-h_1^2)C$$
$$=\tfrac{1}{2}wh(h+2h_1)C$$

and the distance of the point of application of this force from the base of wall

$$x=\frac{h^2+3hh_1}{3(h+2h_1)}$$

P acts through the center of gravity of $ABDE$.

For a wall with an inclined back and with a load on the filling, the pressure (as in the case of a wall with inclined back without any additional loading) is the resultant of P and the weight of the prism of earth 1 ft. in length, the cross-section of which is a trapezoid—for example, $ABFG$, Fig. 12.

4. Objections to Earth Pressure Theories.—The theoretical formulas for earth pressure are considered by many engineers as of little value in designing retaining walls except, perhaps, in special cases. The problem of the retaining wall does not admit of an exact mathematical solution and, since something must be assumed in any case, these engineers would rather assume the thickness at the start than derive this thickness from equations based upon a number of uncertain assumptions. It is pointed out, for example, that all formulas for earth pressure assume that the material to be supported is clean, dry sand—a material that is seldom found in practice—while it is known that all other soils possess considerable cohesion and that they are subject to extensive variation due to moisture, frost, shock, etc., which cannot be included in any formula. Another argument for abandoning earth-pressure formulas is shown in the number of wall failures which have occurred after the walls had stood many years with seeming stability.

5. Methods of Determining Stability of Reinforced-concrete Walls.—The stability of a reinforced-concrete wall as regards overturning may be determined in either of two ways, namely: (1) by using a theoretical formula for the thrust of the earth; or (2) by making the stability against rotation equal to that of a gravity wall that is known to be safe. For reasons already given, the results from the first method may be very misleading unless carefully controlled by the results of experience, while the second method is probably the more satisfactory when a gravity wall has been known to be safe under exactly the same conditions as proposed for the reinforced wall In the second method the overturning resistance of the lightest gravity wall which experience has shown to be safe under the given conditions may

be considered the maximum overturning moment of the earth, and the reinforced-concrete wall may be designed to have an equal stability.

A reinforced-concrete retaining wall must have stability against sliding as well as against overturning. There is very little danger from sliding in gravity walls but, in the case of the reinforced type, the wall is often so light that, unless vertical projections of the base are employed, there is a lack of frictional resistance to withstand the horizontal component of the resultant pressure on the foundation. The resistance offered by the soil in front of the wall increases the stability against sliding, but this amount is usually disregarded as a matter of safety. The friction along the bottom of the wall is equal to the vertical pressure multiplied by the coefficient of friction of concrete on earth. The following values for the coefficient of friction may be safely employed:

COEFFICIENTS OF FRICTION OF CONCRETE ON FOUNDATION MATERIALS

Dry clay	0.50
Wet or moist clay	0.33
Sand	0.40
Gravel	0.60
Wood (with the grain)	0.60
Wood (across the grain)	0.50

For clayey soil it is best to use the value given for wet clay, unless it is reasonably certain that water cannot reach the bottom of the footing.

As with all other structures, a retaining wall should be designed so that the maximum pressure on the soil is within the allowable. This is a much more simple matter with reinforced walls than with the heavy walls of gravity section. There is no likelihood that the concrete will fail by crushing, but an adequate consideration of the maximum pressure on the soil under the foundation should not be neglected on this account. The safe bearing capacity of soils is given approximately in the following table:

SAFE BEARING CAPACITY OF SOILS IN SHORT TONS PER SQUARE FOOT

Kind of material	Minimum	Maximum
Rock, the hardest, in thick layers in native bed	200	....
Rock equal to best ashlar masonry..........	25	30
Rock equal to best brick masonry...........	15	20
Rock equal to poor brick masonry..........	5	10
Clay in thick beds, always dry...............	6	8
Clay in thick beds, moderately dry..........	4	6
Clay, soft................................	1	2
Gravel and coarse sand, well cemented.......	8	10
Sand, dry, compact and well cemented.......	4	6
Sand, clean, dry.........................	2	4
Quicksand, alluvial soils, etc................	0.5	1

The *factor of safety* of a retaining wall is the ratio of the weight of a filling having an angle of internal friction which will just

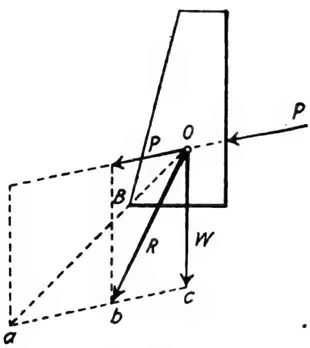


FIG. 13.

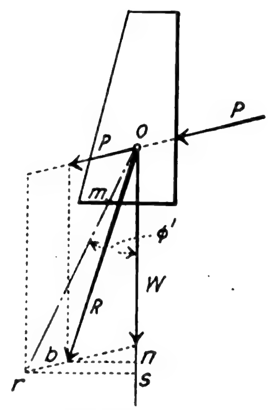


FIG. 14.

cause failure to the actual weight of the filling. There are two factors to be considered, namely: (1) for overturning; and (2) for sliding.

In Fig. 13, let P be the resultant pressure of the earth, and let W be the weight of the wall. Then Ob will be the resultant pressure R tending to overturn the wall. Draw Oa through the point B. For the resultant to pass through B, the wall would be on the point of overturning, and the factor of safety again

overturning would be unity. Denote the factor of safety for the resultant pressure *Ob* as f_{OT}. Then

$$f_{OT}=\frac{ac}{bc}$$

In Fig. 14 construct the angle *nOr* equal to ϕ', the angle of friction of the masonry on the foundation. Now if *R* should pass through *m*, and take the direction *Or*, the wall would be on the point of sliding, and the factor of safety against sliding would be unity. Denote the factor of safety for the resultant pressure *Ob* as f_{SL}. Then

$$f_{SL}=\frac{rs}{bn}$$

6. Walls of Gravity Section.—Trautwine's "Civil Engineer's Pocket-book" gives a table of empirical values which may be used in designing retaining walls of gravity section for average conditions. The values were obtained from a set of experiments on small wooden models and have been used with good results in engineering practice. There are no values given for plain concrete walls but they may safely be assumed equivalent to those of cut stone laid in mortar. In the following table (taken from the book above mentioned) for cut-stone masonry, the earth is assumed to slope up from the top of the wall till it reaches a level at the height indicated by the ratio is the first column.

Ratio of height of earth to height of wall above ground	Ratio of thickness of base to total height of wall $=\frac{t}{h}$	Ratio of height of earth to height of wall above ground	Ratio of thickness of base to total height of wall $=\frac{t}{h}$
1.0	0.35	2.0	0.58
1.1	0.42	2.5	0.60
1.2	0.46	3.0	0.62
1.3	0.49	4.0	0.63
1.4	0.51	6.0	0.64
1.5	0.52	9.0	0.65
1.6	0.54	14.0	0.66
1.7	0.55	25.0	0.68
1.8	0.56	or more	

Trautwine recommends that when the backing is somewhat consolidated in horizontal layers, each of these thicknesses may be

reduced, but that no rule can be given for this. He also states that since sand and gravel have no cohesion, the full dimensions as above should be used with these materials, even though the backing be deposited in layers. A mixture of sand, or earth with a large proportion of pebbles, bowlders, etc., will exert a greater pressure against the wall than the materials ordinarily used for backing; and hence when such backing has to be used, the above thicknesses should be increased, say, about $\frac{1}{4}$ to $\frac{1}{6}$ part.

The values in the above table apply especially when the batter of the face of the retaining wall is limited (as is customary) to $1\frac{1}{2}$ in. to the foot, and when the back is vertical. Mr. Trautwine says—"After a wall *abco*, Fig. 15, with a vertical back has been proportioned by the above rule, it may be converted into one with an offsetted back, as *aino*. This will present greater resistance to overturning, and yet contain no more material. Thus, through the center *t* of the back, draw any line *in;* from *n* draw *ns* vertical; divide *is* into any even number of equal parts (in the figure there are 4); and divide *sn* into one more equal parts (in the figure there are 5). From the points of division draw horizontal and vertical lines, for forming the offsets, as in the figure."

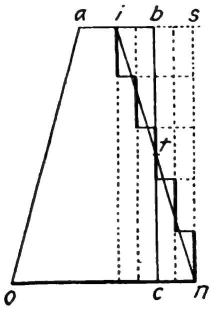


Fig. 15.

7. Equivalent Fluid Pressure for Gravity Walls.—In order to make the stability of a reinforced-concrete wall, as regards overturning, equal to that of a gravity wall, it will be convenient to determine the fluid pressure under which an equivalent gravity wall will be stable and then apply this pressure to the reinforced type. Since the magnitude, point of application, and line of action of fluid pressure is much more accurately known than that of earth, results by this method are likely to be in most cases a great deal nearer the truth than if use is made of the theoretical formulas.

As shown in Art. 2, the total pressure on a wall due to a liquid is given by the formula

$$P = \tfrac{1}{2}wh^2$$

The fluid pressure P and the weight of wall W, Fig. 4, cause a force R to act upon the base of wall. The distribution of pressure on the base due to the resultant acting at different points

will be explained. Consider the base represented in projection by *EF*, Fig. 16. When *R* acts at the center of gravity *O*, the intensity of pressure is uniform over the section and is equal to the vertical component of *R* divided by the area of section, or $\frac{W}{A}$. If *R* acts at any other point, as *N*, the force *W* is equivalent to an equal *W* at *O* and a couple whose moment is Wx_0. At any point distant x from *O* the intensity of the pressure due

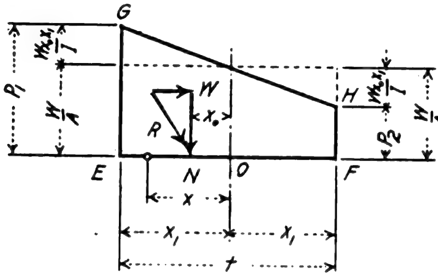


FIG. 16.

to this moment is (by the common flexure formula for homogeneous beams) $\frac{Wx_0x}{I}$, in which *I* is the moment of inertia of the section about an axis through *O* at right angles to the plane of the paper. At the edges *E* and *F* this intensity $= \frac{Wx_0x_1}{I}$. The intensity of pressure at edge *E* is

$$p_1 = \frac{W}{A} + \frac{Wx_0x_1}{I} \tag{1}$$

and at edge *F* it is

$$p_2 = \frac{W}{A} - \frac{Wx_0x_1}{I} \tag{2}$$

In gravity walls it is necessary to limit the value of x_0 in order to ensure that at no horizontal section near the base will part of the stress be tensile as, without the aid of steel, masonry cannot be depended upon to resist tension. This limiting value of x_0 is found by equating p_2 in the second equation to zero, or

$$\frac{W}{A} - \frac{Wx_0x_1}{I} = O$$

$$x_0 = \frac{I}{Ax_1}$$

If t is taken as the width of the base and l the length at right angles to the plane of the paper, then

$$A = lt, x_1 = \frac{t}{2}, \; I = \frac{lt^3}{12}$$

and the limiting value of $x_0 = \dfrac{lt^3}{(12)(lt)\left(\frac{t}{2}\right)} = \dfrac{t}{6}$. As x_0 may be on either side of O, it follows that the resultant thrust must fall within the middle third of the base in order that there may be no tensile stress on any section near the base. When $x_0 = \frac{t}{6}$, the value of the intensity of stress varies from $2\left(\frac{W}{A}\right)$ at the edge nearest the resultant to zero at the opposite edge (Fig. 17).

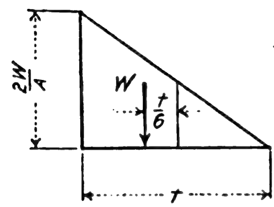


FIG. 17.

In view of the above, it would seem practicable to determine the fluid pressure under which a gravity wall will be stable by considering the resultant of all forces above the base to intersect the base at the outer edge of the middle third. If, then, the reinforced wall is designed so that it will be equally stable, the two walls will be practically equivalent.

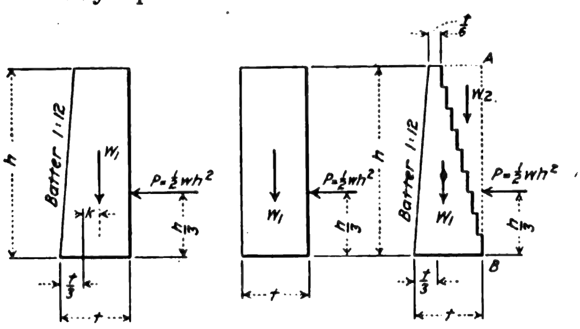


FIG. 18.

Calculations will now be given for the three principal forms of gravity walls which are represented in Fig. 18. Form (b) is used for relatively low walls.

Form (a). The batter of the front face will be taken as 1 in

per foot, or 1 : 12. The weight of the masonry will be assumed at 150 lb. per cubic foot. Let P denote the resultant fluid pressure per linear foot of wall acting at a distance $\frac{1}{3}h$ above the base. Let w denote the weight per cubic foot of such fluid. Then $P = \frac{1}{2}wh^2$. Let W_1 denote the weight of masonry per linear foot.

Assume that the resultant of P and W_1 intersects the base at the edge of the middle third. Then, equating moments about this point, we have

$$\frac{1}{2}wh^2\left(\frac{h}{3}\right) = W_1k$$

or

$$w = 150\left(\frac{t^2}{h^2} + \frac{1}{12}\frac{t}{h} - \frac{1}{144}\right) \tag{3}$$

Form (b). Taking moments as before about the edge of the middle third

$$w = 150\frac{t^2}{h^2} \tag{4}$$

Form (c). The weight of the earth will be assumed at 100 lb. per cubic foot. The batter of the front face will be taken at 1 : 12 and the top width at one-sixth of the bottom width. It will be assumed that the fluid pressure acts against a vertical plane AB and the stability of volume to the left of this plane, including the weight of the earth, needs to be considered.

Equating moments as before,

$$w = 150\left(\frac{73t^2}{108h^2} + \frac{5t}{54h} - \frac{1}{216}\right) \tag{5}$$

To show the relative stability of the three forms, the following values have been determined for w, or equivalent fluid weight, by substituting various values of $\frac{t}{h}$ in the above equations:

	$\frac{t}{h}$	w
Form (a)	$\frac{1}{3}$	19.8
	$\frac{4}{10}$	28.0
	$\frac{1}{2}$	42.7
Form (b)	$\frac{1}{3}$	16.7
	$\frac{4}{10}$	24.0
	$\frac{1}{2}$	37.5
Form (c)	$\frac{1}{3}$	15.2
	$\frac{4}{10}$	21.1
	$\frac{1}{2}$	31.6

It should be noted that the fluid pressures as above determined are not necessarily the actual earth pressures, but may be used in the design of reinforced-concrete walls to secure stability practically equivalent to corresponding walls of gravity section. The formulas derived may be used for all the usual or more common types of retaining walls since considerable variation in dimensions produces only a slight effect on the resultant fluid pressure. A fluid weight of 25 lb. per square foot may be taken as a basis of design under average conditions.

CHAPTER II

DESIGN

8. Principal Types of Reinforced-concrete Walls.—There are two principal types of reinforced-concrete retaining walls, namely: (1) the cantilever type; and (2) the counterforted type. Fig. 19 represents in outline the usual or cantilever type which is suitable for low walls. It consists of a vertical stem *AC* attached to a footing *BD*. The upright portion resists the thrust of the earth by virtue of its strength as a cantilever beam, and the parts *BK* and *CD* act in a like manner. For larger walls the counterforted type is employed, the part *AC* being tied to *CD* at intervals by back walls *ACD* in the form of narrow transverse walls with tension reinforcement called counterforts. Sometimes the portion *BK* is connected to the vertical wall *AC* by means of buttresses.

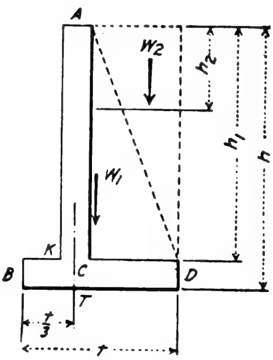


Fig. 19.

In buildings, basement walls to retain earth may consist of light walls reinforced vertically from basement floor to the first floor, or reinforced horizontally from column to column. Such walls are designed as simply supported slabs—the reactions being taken by the basement and first floors, or by the columns. These walls may be reinforced to carry themselves from footing to footing, thus requiring no foundations of their own. This type of wall will be treated under *Buildings* and will receive no further consideration here.

9. General Method of Procedure.—The general method of procedure in the design of reinforced-concrete retaining walls is very much the same whether the stability against rotation is determined by using a theoretical formula for the thrust of the earth, or whether, by employing an equivalent fluid pressure, the stability is made equal to that of a gravity wall that is known

to be safe. The procedure will be outlined only for the latter method of determining stability, but what is given below will also apply when the earth pressure formulas are used, except that, where the earth thrust does not act horizontally, the vertical component of the thrust must be considered when determining the resultant pressure on the base of wall. A design of a retaining wall using Rankine's formula for earth pressure will be given later. This will be done so as to leave no doubt in the mind of the student as to the method to be followed in such cases.

An investigation of the two types of reinforced-concrete retaining walls shows that a minimum amount of material is required when the middle-third point is approximately under the center of the vertical wall—namely, at T, Fig. 19. This minimum, however, does not show a great saving over the material required with lengths of BT other than $\frac{t}{3}$. An investigation also shows that the stability of the reinforced wall for a given value of $\frac{t}{h}$ is about the same as a solid wall of form (c) (Fig. 18), having the same value of $\frac{t}{h}$.

Investigation also shows that minimum soil pressure occurs when the stem AC is in the center of BD, and that maximum pressure occurs when BK is zero. On the other hand, sliding stability is found to decrease as BK increases.

From the above facts, it appears the best plan, if the position of the stem with respect to the base is not rigidly fixed, and if other conditions do not control, to start a design with a value $\frac{t}{h}$ for the reinforced wall identical to that of a solid wall of form (c), which withstands the same fluid pressure as required for the reinforced type, and to make the distance BT about equal to $\frac{t}{3}$. A complete table for form (c) of gravity walls, giving values of w for various values of $\frac{t}{h}$, is convenient if the above-mentioned method of design is followed. Such a table is given below, based on a level back-fill, the values being derived from equation (5) of Art. 7 in a similar manner to the short table previously given. In a wall with sloping back-fill, a somewhat greater trial value of $\frac{t}{h}$ should be taken than is given by table—

that is, of course, considering h as the height of masonry. For a loaded back-fill, a trial value of $\frac{t}{h}$ may be determined by considering the height h in the table to include the height of an equivalent surcharge of earth .

$\frac{t}{h}$	w	$\frac{t}{h}$	w
0.30	12.6	0.50	31.6
0.35	16.6	0.55	37.6
0.40	21.1	0.60	44.1
0.45	26.1	0.65	51.2

After trial values for the length of base and length of projection BC are decided upon (generally after rough computations are made for stability), the thickness of footing should be assumed. The forces to be considered upon the three parts of the wall—namely, AC, BK, and CD—should next be determined. The resultant force on the wall AC may be taken as a horizontal force equal to $P_1 = \frac{1}{2} wh^2_1$ and applied a distance $\frac{1}{3} h_1$ above the top of footing. On any length h_2 from the top of wall, the resultant force is likewise $\frac{1}{2} wh^2_2$ and is applied at a distance $\frac{2}{3} h_2$ below the top. The pressure per square foot at any point on the wall a distance h_2 below the top is equal to wh_2.

If we let W_1 denote the weight of masonry per linear foot and W_2 the weight of earth per linear foot above the floor CD, Fig. 19, then the total pressure on the base will equal the total weight W_1+W_2 and it will be applied at a distance $\frac{1}{3}t$ from point B if the resultant of all forces intersects the base as planned—that is, at the outer edge of the middle third. The average unit pressure will be $\frac{W_1+W_2}{t}$, and the maximum pressure at B will be twice this value. The pressure at D will be zero. The maximum unit pressure on the base should be within the safe bearing power of the soil beneath, otherwise the length of the base should be increased. The length should also be increased if the tendency to slide is too great or if the resultant pressure on the base does not come within the middle third.

When determining the point of application of the resultant force on the base, or when determining the tendency of the wall

to slide, the total fluid pressure $P=\frac{1}{2}wh^2$ (Fig. 19) should be considered. If this resultant of all forces intersects the base within the third point, the exact distribution of earth pressure along the base should be determined. Formulas (1) and (2) of Art. 7 give the intensity of pressure at each end of the base if the proper values are substituted.

$$A=lt,\ x_1=\frac{t}{2},\ I=\frac{lt^3}{12},\ l=1\text{ ft.},$$

and the equations above mentioned reduce to the following:

$$p_1=\frac{W}{t}\left(1+\frac{6x_0}{t}\right)$$

$$p_2=\frac{W}{t}\left(1-\frac{6x_0}{t}\right)$$

Referring to Fig. 20, it is clear that a *trapezoid* of pressure acts upward against the cantilever BK. The center of gravity of the trapezoid should be found and the resultant applied at that point. The only downward force to consider is the weight of the cantilever itself, since the earth back of the wall may be

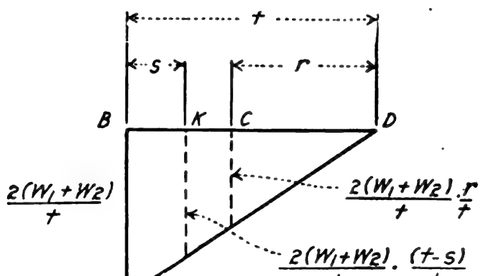


FIG. 20.

put in place before the outer footing is covered. In addition to its own dead weight, there are two forces acting on *CD;* namely, the weight of the earth W_2 above the floor and the upward pressure against the base. This latter pressure varies as shown in Fig. 20.

The complete wall (including the counterforts in the counterforted type) may now be proportioned. When a radical change is made in the shape of the footing from that assumed, good judgment should be exercised as to whether or not it is necessary

to determine the new intersection of the resultant pressure on the base. Of course, the intersection should lie within the middle third for the final design.

It is customary to build retaining walls of plain concrete with vertical contraction joints 40 to 50 ft. apart, this distance varying according to the thickness of the wall. In reinforced walls, however, if sufficient reinforcement (0.2 to 0.4 per cent) is employed and placed near the surface, contraction cracks may be forced to take place at such frequent intervals as to be quite invisible—at least for considerable length of wall. There seems to be a difference of opinion as to whether or not shrinkage joints are ever needed in reinforced-concrete structures. The report of the Joint Committee (see Appendix) should be read in this connection.

10. Required Thickness of Concrete Covering for Steel.—Experimental tests have proved conclusively that 1½ to 2 in. of well-mixed concrete, made in ordinary proportions and mixed wet, will effectively prevent the corrosion of steel under extraordinary conditions—conditions much more severe than occur in the ordinary retaining wall or building. For example, steel protected by an inch or more of sound concrete has been found unaffected when subjected continuously to the action of steam, air, and carbon dioxide for a period of three weeks. Unprotected pieces of steel exposed to the same test "were found to consist of rather more rust than steel." Tests have been made on loaded beams as well as on concrete unstressed, and it has been proved that there is no danger of rusting the steel through the cracks formed in the concrete under tension, until nearly the breaking point of the steel is reached. No rust has been found even on steel stressed to its elastic limit.

Since we have the knowledge derived from the tests above mentioned, there would seem to be no good reason for allowing more than 1½ to 2 in. as a protective covering for the steel in retaining-wall design—especially when the wall is to be well drained, and the concrete operations well superintended. In the designs which follow, this condition of affairs will be assumed. In cases where close inspection of the work is not expected, a greater depth of covering should be provided—say, 2 to 4 in.—depending on the part of wall under consideration. Where steel is to be placed near the base of footing 2 in. should be considered the minimum thickness of covering for the very best soil conditions.

In designs where conditions are not accurately known, at least 3 in. should be allowed.

11. Design of Cantilever Type.—The design of the cantilever type of reinforced-concrete retaining walls resolves itself into the design of three cantilevers, each cantilever acted upon by approximately a uniformly varying load. This type of reinforced wall is not economical for heights greater than about 20 ft.

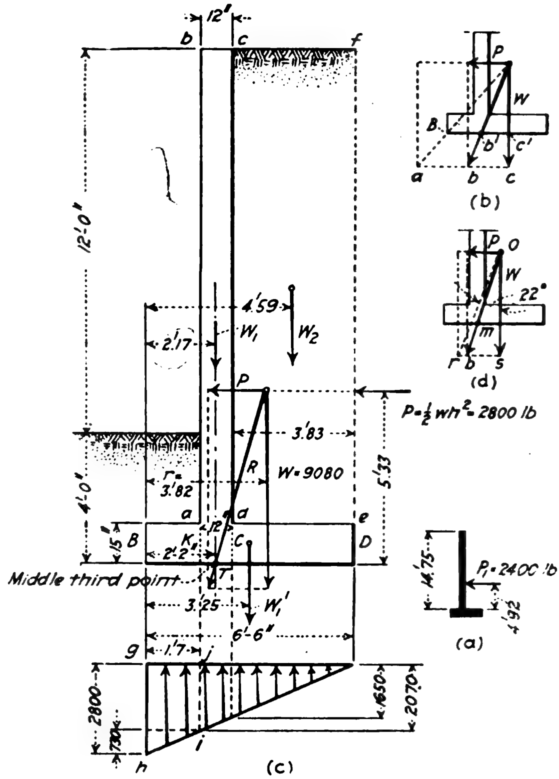


Fig. 21.

The notation which will be employed is essentially that recommended by the Joint Committee. (See Appendix.)

$\boxed{\text{Design No. 1.}}$ Design a retaining wall to restrain a level earth bank 12 ft. in height. Assume the base of footing to be embedded 4 ft.; in other words, assume that this [illegible] will protect the foundation from the effects of frost. M[illegible] allow-

able pressure on the soil is 2 short tons per square foot. Coefficient of friction of concrete on earth = 0.40 (angle of friction corresponding, 22° approximately). Assume earth to weigh 100 lb. per cubic foot and the concrete 150 lb. per cubic foot. A 2000-lb. concrete with proportions 1:2:4 will be employed with the same working stresses as recommended by the Joint Committee. Plain round rods of medium steel will be assumed.

It will also be assumed that the position of the stem with respect to the base is not rigidly fixed, that Trautwine's empirical rules have been previously followed in plain concrete wall design under similar conditions to those at hand, and that these rules have given satisfactory results. In Art. 6 we find that Trautwine recommends 0.35 as the ratio $\frac{t}{h}$ for a level backfill and for a form of wall designated as (a) in Fig. 18. Referring to the table in Art. 7, a value of $w=21.9$, say 22, is found to correspond to the given ratio of $\frac{t}{h}$ for the (a) form of wall. This value of w corresponds to a value of $\frac{t}{h}=0.41$ for the (c) form, which we have found is a form of gravity wall that we may use for comparison with the reinforced type. Since $h=16$ ft., t will be tried at (0.41) (16) = 6.56 ft., say $6\frac{1}{2}$ ft. (See Fig. 21.) BT will be made $\frac{1}{3}t$ or 2 ft. 2 in. The thickness of the footing cannot be determined in advance but for the present it will be assumed at 15 in.

Vertical Wall.—Assuming a 15-in. thickness for the footing gives a height of wall above the top of footing of 14 ft. 9 in. The total pressure on this vertical stem $=\frac{1}{2}w(14.75)^2=\frac{1}{2}(22)(14.75)^2=2400$ lb. and is applied at a distance $\frac{1}{3}(14.75)=4.92$ ft. above the bottom of the stem. (See Fig. 21*a*.) Thus the bending moment at the top of the footing, $M=(2400)(4.92)(12)=141{,}700$ in.-lb. From Table 2 of Volume I for $f_c=650$ and $f_s=16{,}000$, we obtain $K=107.4$. Then,

$$bd^2=\frac{141{,}700}{107.4}=1320$$

T [illegible] in., the resulting thickness of vertical slab at the b [illegible] n. plus $1\frac{1}{2}$ in. (as a minimum to properly imbed t [illegible] tal thickness of 12 in. Table 2 shows $p=0.0077$;

hence, area of steel $a_s = (0.0077)(12)(10.5) = 0.97$ sq. in. per linear foot of wall. Table 4 shows that $\frac{3}{4}$-in. round rods spaced $5\frac{1}{2}$ in. on centers will give the required area. These rods, of course, should be placed vertically along the back of the wall.

The bending moments in the vertical beam vary as the cubes of the distances below the top while the resisting moments vary as the squares of the thickness of the beam. Thus, if a beam should have the required thickness at the top of the footing and should taper uniformly to zero thickness at the top of wall, it would have more than the required strength at intermediate points. However, considerable thickness is needed at the top to resist shocks, frost, etc., and for convenience in placing the concrete. For simplicity the top thickness in this design will be taken as 12 in., although 10 in. may be quite sufficient.

At a point 10 ft. below the top of wall, the bending moment is $\frac{1}{2}(22)(10)^2\left(\frac{10}{3}\right)(12) = 44{,}000$ in.-lb., and at 5 ft. below it is $\frac{1}{2}(22)(5)^2\left(\frac{5}{3}\right)(12) = 5500$ in.-lb. The corresponding values of K are $\frac{44{,}000}{(12)(10.5)^2} = 33.3$ and 4.2. Referring to Table 3, it will be seen that less than $\frac{1}{3}$ and $\frac{1}{6}$ of the reinforcement used at the footing ($p = 0.0077$) will be sufficient at these points respectively. For example, the p corresponding to $K = 33.3$ is 0.0023, while $\frac{1}{3}$ of $0.0077 = 0.0026$. Although the table does not include a value for K as low as 4.2, it can be readily shown that using a rough value for j in the formula $p = \frac{K}{f_s j}$, a value of p will be obtained which will be considerably smaller than $\frac{1}{6}$ of $0.0077 = 0.0013$. Thus if every third rod is carried up to within 5 ft. of the top and one out of every six to the top of the wall, the vertical wall is sure to be safely designed.

Maximum shear on the vertical stem occurs [illegible] footing and equals 2400 lb. The corresponding u[illegible]

$$v = \frac{V}{bjd} = \frac{2400}{(12)(0.874)(10.5)} = 22 \text{ lb. per squa[illegible]}$$

which may easily be carried by the concrete, and th[illegible] are needed.

Since bond stress varies as shear, the m[illegible] also occurs at the bottom of the vertical stem.

$$u=\frac{V}{\Sigma ojd}=\frac{2400}{\left(\frac{12}{5.5}\right)(2.36)(0.874)(10.5)}=51 \text{ lb. per square inch}$$

and thus the bond stress is within the allowable.

Each vertical rod should have a length sufficient to prevent pulling out and Table 16 shows this length to be $50\times\frac{3}{4}=37\frac{1}{2}$ in. (The Joint Committee recommends a distance somewhat greater than this. See report at end of this volume.) Thus, the vertical rods must extend at least about 38 in. into the footing, or be anchored in a substantial manner. The anchorage may be provided by bent ends; by looping the rods around anchor-bars near the bottom of the footing; or, when a projection is needed below the footing to prevent sliding, this projection may be so placed as to aid materially in providing length of grip for the vertical rods. Rods up to 1 in. in size are readily bent to any desired angle and it is common design to obtain the grip of the vertical rods by bending them into the outer cantilever where they may serve as horizontal reinforcement.

Longitudinal rods should be inserted in the vertical wall to prevent large cracks. Cracking is more liable to occur near the top and on the outside, so these parts should be the more heavily reinforced against shrinkage and temperature stresses. About 0.4 of 1 per cent of horizontal steel will be employed, based on the total cross-section of the concrete and approximately two-thirds of this will be placed on the front wall. (See Art. 27 of Volume I.) The Joint Committee recommends an amount of reinforcement generally not less than one-third of 1 per cent.

$$a_s=(0.004)(12)(12)=0.58 \text{ sq. in.}$$

Table 4 shows a 4-in. spacing sufficient for $\frac{1}{2}$-in. round rods. This average will be maintained if the spacing be 6 in. in front and 12 in. at the back of the wall.

Footing.—Before the trial footing can be investigated in order to determine whether or not it has a length sufficient to give satisfactory stability to the wall, the total pressure acting upon the bottom of the footing should be computed—also the point of application of this pressure. Referring to Fig. 21, let

W_1 denote weight of concrete represented by area *abcd.*

W_2 denote weight of earth represented by area *cfed.*

W_1' denote weight of concrete in footing.

Then,

$W_1 = (14.75)(150) = 2210$ lb.
$W_2 = (3.83)(14.75)(100) = 5650$ lb.
$W_1' = (6.5)(1.25)(150) = 1220$ lb.

Now the distance from the toe B to the line of action of the resultant vertical force W (considering the weight of the earth above the outer cantilever as negligible) may be found by multiplying the distance from B to the center of gravity of each area by the respective weight and dividing the sum of such moments by the sum of the weights. Thus

$$r = \frac{(2210)(2.17)+(5650)(4.59)+(1220)(3.25)}{2210+5650+1220} = \frac{34,710}{9080} = 3.82 \text{ ft.}$$

and $W = 9080$ lb. The line of pressure drawn for P and W intersects the base practically at the middle third point as planned.

Fig. 21(b) shows that the factor of safety against overturning (see Art. 5)

$$f_{OT} = \frac{ac}{bc} = 2.3, \text{ or } f_{OT} = \frac{Bc'}{b'c'} = \frac{3.82}{1.65} = 2.3$$

A factor of 1.5 to 2 may usually be considered as ample for reinforced walls because the stability of wall is increased by the resistance of earth to shear along the line ef (Fig. 21) and by the passive pressure of the filling in front of the wall when this earth is put in place before that behind the wall.

The average pressure per square foot on the foundation is $\frac{9080}{6.5} = 1400$ lb., and the maximum pressure at B is twice this value, or 2800 lb., which is within the allowable. The pressure at D is zero. Fig. 21(c) shows the distribution of pressure throughout the length of the base.

Fig. 21(d) shows the factor of safety against sliding (see Art. 5) to be

$$f_{SL} = \frac{rs}{bs} = 1.3, \text{ or } f_{SL} = \frac{(\text{Coeff. of friction})(W)}{P}$$
$$= \frac{(0.4)(9080)}{2800} = 1.3$$

It should be remembered that this factor of safety does not take into account the fact that resistance is offered against sliding nor the fact that if the wall slides, the cohesion of the earth along

the line *ef* (Fig. 21) must be destroyed. For the above reasons the wall will be considered safe against a sliding failure.

Inner Cantilever.—Fig. 21 shows the length of this cantilever to be 3.83 ft. There are three forces acting—the upward pressure of the soil and the downward weights of the cantilever and earth filling. At C the shear

$$V=5650+(3.83)(1.25)(150)-\frac{1650}{2}(3.83)$$
$$=5650+720-3160=3210 \text{ lb.}$$

and the moment at the same section

$$M=\left[(5650+720)\left(\frac{3.83}{2}\right)-(3160)\left(\frac{3.83}{3}\right)\right]12$$
$$=(12{,}200-4030)\ (12)=98{,}000 \text{ in.-lb.}$$

and is negative.

The minimum depth to steel by moment is (Table 2)

$$\sqrt{\frac{98{,}000}{(12)(107.4)}}=9 \text{ in.}$$

This depth is undoubtedly too small to satisfy the bond stress which is considered below.

The approximate depth to satisfy bond stress may always be obtained by using the average value $\frac{7}{8}$ for j. From the formula $u=\frac{V}{\Sigma ojd'}$, we have $d'=\frac{V}{\Sigma ouj}$. Taking $\frac{3}{4}$-in round rods, same size as employed in the vertical stem, and assuming an $8\frac{1}{4}$-in. spacing ($1\frac{1}{2}$ times the spacing of the vertical rods), we have as the approximate depth to satisfy bond stress

$$d'=\frac{3210}{\left(\frac{12}{8.25}\right)(2.36)(80)(\frac{7}{8})}=13\frac{1}{2} \text{ in.}$$

If we use $d=13\frac{1}{2}$ in. (total depth 15 in.), then $p=\frac{0.442}{(8.25)(13.5)}=$ 0.0040 and Table 3 gives $j=0.903$. The resulting bond stress

$$u=\frac{3210}{\left(\frac{12}{8.25}\right)(2.36)(0.903)(13.5)}=77 \text{ lb. per square inch}$$

which is less than the allowable.

$$v=\frac{3210}{(12)(0.903)(13.5)}=22 \text{ lb. per square inch}$$

and no stirrups are necessary. Rather than use stirrups which

are troublesome in construction, it is better in nearly every case to increase the thickness.

The stress in the steel

$$f_s = \frac{M}{pjbd^2} = \frac{98,000}{(0.0040)(0.903)(12)(13.5)^2} = 12,400 \text{ lb. per sq. in.}$$

and the length of embedment necessary to prevent slipping

$$\frac{fi}{4u} = \frac{(12,400)(\frac{3}{4})}{(4)(80)} = 30 \text{ in.}$$

The stress in the concrete is clearly less than the allowable since the moment depends upon the steel.

It is seen from the above computations that bond is the controlling element. Since bond varies as shear, a shear diagram may be used to determine how much the footing may be tapered toward the inner end. Fig. 22 is such a diagram, with ordinates plotted at 1-ft. intervals. The end of the footing cantilevers

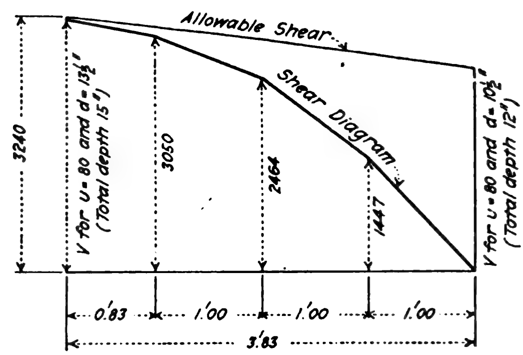


Fig. 22.

should have a minimum practical thickness of at least 12 in., since inequalities in the character of the soil may increase the unit pressures at some points. The problem, then, is to determine whether the footing may be tapered to this thickness at the end; if not, a greater end depth must be taken. The amount of shear needed at each end of the cantilever to cause u to equal its allowable value should next be determined, and the ordinates representing these values should be plotted to the same scale as the rest of the diagram. A straight line joining the upper ends of these ordinates represents the allowable shear throughout the beam considering the amount of steel as constant.

The ordinates to the allowable shear curve may be found by transposing the bond formula $u=\frac{V}{\Sigma ojd}$ into the form $V=u\Sigma ojd$, and substituting for u its allowable value. Thus the ordinate at the left in Fig. 22 is found, accurately enough, as follows:

$$V=(80)\left(\frac{12}{8.25}\right)(2.36)(\tfrac{7}{8})(13.5)=3240 \text{ lb.}$$

Outer Cantilever.—The computations for the outer cantilever are similar to those for the inner cantilever. Fig. 21 shows the length of this cantilever to be 1.7 ft. Since the earth back of the wall may be put in place before the outer footing is covered, the weight of earth above this part of the footing will be neglected. The downward weight is that of the cantilever itself and the upward pressure is represented by the trapezoid *ghij* in Fig. 21(c).

The downward weight is $(1.25)(1.7)(150)=319$ lb. with a lever arm about the front of the vertical wall of 0.85 ft.; moment corresponding $=(319)(0.85)(12)=3250$ in.-lb. The moment of the upward forces about the same section is

$$\left[(2070)(1.7)\left(\frac{1.7}{2}\right)+(730)\left(\frac{1.7}{2}\right)\left(\frac{2}{3}\right)(1.7)\right]12=44{,}300 \text{ in.-lb.}$$

The total moment

$$M=44{,}300-3250=41{,}050 \text{ in.-lb.}$$

Referring to Table 2, the minimum depth to steel (for moment) at the front of the vertical wall should be

$$\sqrt{\frac{M}{bK}}=\sqrt{\frac{41{,}050}{(12)(107.4)}}=6 \text{ in.}$$

and the area of steel $a_s=(0.0077)\ (12)\ (6.0)=0.55$ sq. in. The depth to satisfy bond stress or shear will undoubtedly control. It is common design to bend the vertical rods into the horizontal to form the reinforcement of the outer cantilever. If all the steel in the vertical wall is used in this way, we should have 0.97 sq. in., which is more than sufficient as regards moment.

$$V=\frac{2800+2070}{2}(1.7)-319=3820 \text{ lb.}$$

Assuming $\frac{7}{8}$ for j, we have as the approximate depth to satisfy bond stress

$$d=\frac{3820}{\left(\frac{12}{5.5}\right)(2.36)(80)(\frac{7}{8})}=11 \text{ in.}$$

If we use $d=13$ in. with a total depth of 15 in. (2 in. protective covering at the bottom of footing should be considered a minimum), then $p=\frac{0.97}{(12)(13)}=0.0062$ and Table 3 shows $j=0.884$. The resulting bond stress

$$u=\frac{3820}{\left(\frac{12}{5.5}\right)(2.36)(0.884)(13)}=65 \text{ lb. per square inch}$$

which is satisfactory. The stress in the steel

$$f_s=\frac{M}{pjbd^2}=\frac{41{,}050}{(0.0062)(0.884)(12)(13)^2}=3700 \text{ lb. per square inch}$$

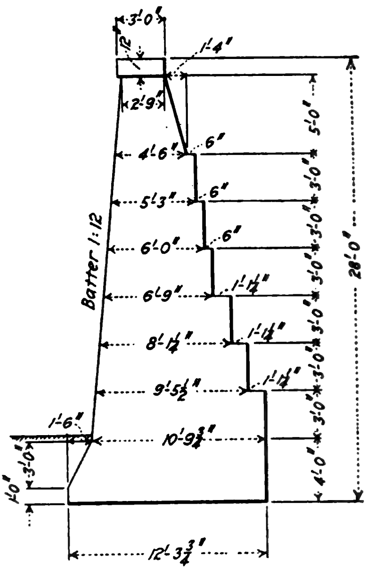


FIG. 23.

and the length of embedment necessary to prevent slipping

$$\frac{fi}{4u}=\frac{(3700)(\frac{3}{4})}{(4)(80)}=9 \text{ in.}$$

This distance can be easily secured.

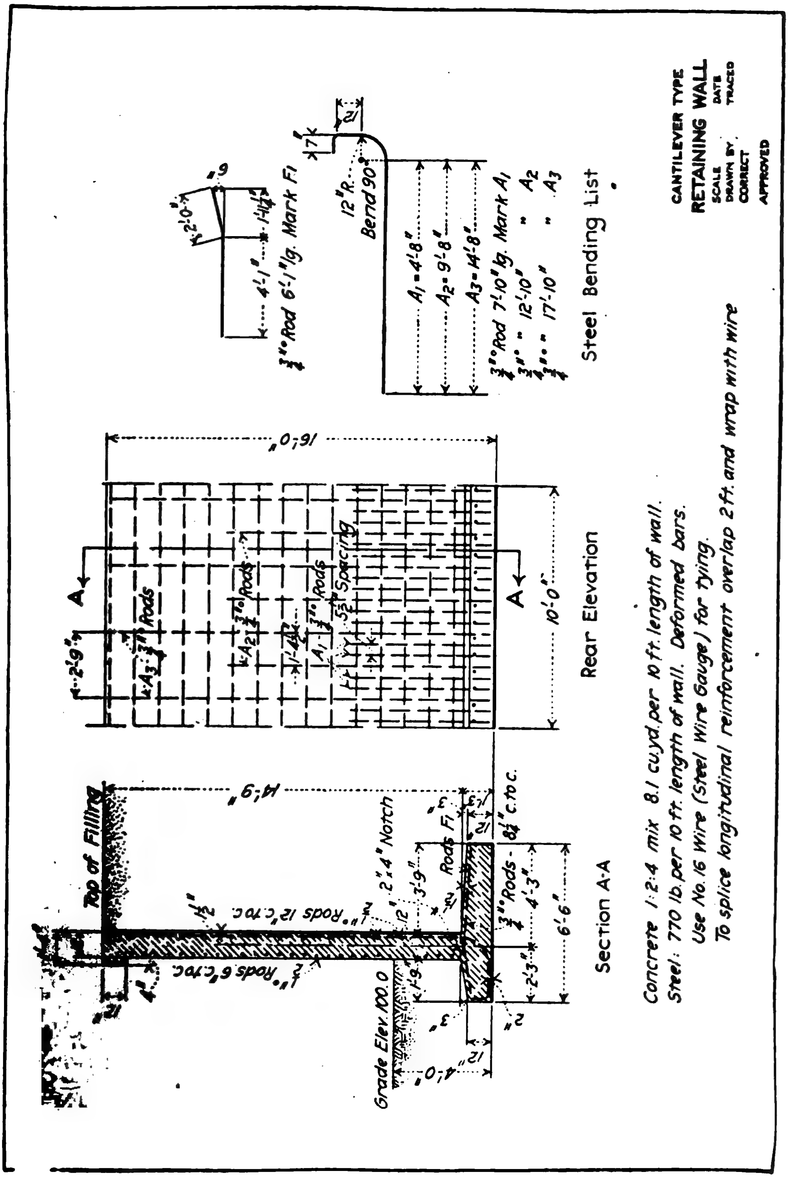
The maximum unit shear involving diagonal tension

$$v=\frac{3820}{(12)(0.884)(13)}=28 \text{ lb. per square inch}$$

and no stirrups are needed.

Plate I gives the complete design of wall. The footing poured first and, after this becomes hard, the forms are erected

PLATE I


CANTILEVER TYPE
RETAINING WALL
SCALE
DRAWN BY
CORRECT
APPROVED
DATE
TRACED
Concrete 1:2:4 mix 8.1 cu.yd. per 10 ft. length of wall.
Steel: 770 lb. per 10 ft. length of wall. Deformed bars.
Use No. 16 Wire (Steel Wire Gauge) for tying.
To splice longitudinal reinforcement overlap 2 ft. and wrap with wire
Section A-A
Rear Elevation
Steel Bending List
Top of Filling
Grade Elev. 100.0
2"x4" Notch
14'-9"
16'-0"
10'-0"
6'-6"
4'-3"
2'-3"
3'-9"
1'-9"
4'-0"
Rods F1
Rods 12" c. to c.
Rods 6" c. to c.
Spacing
12" R.
Bend 90°
A1 = 4'-8"
A2 = 9'-8"
A3 = 14'-8"
Rod 6'-1" lg. Mark F1
Rod 7'-10" lg. Mark A1
12'-10" A2
17'-10" A3

for the vertical wall. A 2-in. by 4-in. notch should be made in the footing, as shown in Plate I, so that in case there is no bond between the vertical wall and the footing, the direct shear may be properly provided for. Rough computations, using one-half the allowable compressive strength of the concrete as the allowable stress in direct shear, show that the notch is more than ample even when friction and the bearing value of the rods are disregarded.

PROBLEM

1. A cantilever reinforced-concrete retaining wall is to be designed to have the same stability as the standard plain-concrete wall of the New York Central and Hudson River Railroad, shown in Fig. 23. Take 16 ft. as the height of wall above ground and design the wall assuming all other data (except w) the same as in the preceding example. Make the ends

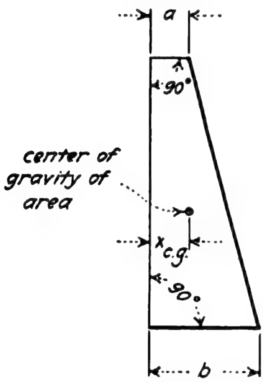


FIG. 24.

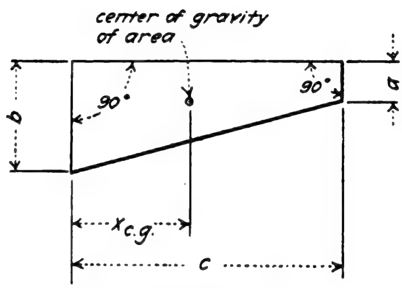


FIG. 25.

of the three cantilevers 12 in. in thickness and design the wall proper with a vertical front face and with a uniform batter at the back. Assume the trial thickness of base at 18 in. In the design of the reinforced wall a fluid should be considered which weighs the next larger whole number of pounds than that obtained from the Standard New York Central wall. In finding weights and centers of gravity in Fig. 23 work about a vertical plane passing through the rear end of footing. Submit all computations, also the completed design neatly drawn in ink.

Note.—It is convenient in retaining wall design to have the following formulas at hand:

$$x_{c.g.} \text{ in Fig. 24} = \frac{a^2+ab+b^2}{3(a+b)}.$$

$$x_{c.g.} \text{ in Fig. 25} = \frac{c}{3} \cdot \frac{2a+b}{a+b}$$

$\boxed{\text{Design No. 2.}}$ Using Rankine's formula for earth pressure, design a retaining wall to be 15 ft. high above the ground and to support an earth bank whose surface has an upward 30° slope from the top of wall. Consider the safe bearing power of the soil at 3 short tons per square foot. Angle of internal friction of the earth filling is 35°. Coefficient of friction of the concrete upon the earth foundation is 0.40. The earth filling weighs 100 lb. per cubic foot and the concrete 150 lb. per cubic foot. A $1:2\frac{1}{2}:5$ concrete is to be employed with $f_c=500$ lb., $f_s=16{,}000$, $u=80$, and $v=30$ lb. per square inch; also $n=15$. Some type of round deformed bar will be assumed in order to allow 80 lb. per square inch in bond.

It will also be assumed that the foundation is to be 3 ft. below the ground surface and that the face of the wall is to be brought as close as possible to a given property line.

The length of the base will be tried at 9 ft. 8 in. and the thickness of the footing will be assumed at 30 in.

Vertical Wall.—The height of the vertical wall assuming a 30-in. footing is $15\frac{1}{2}$ ft. (See Fig. 26.) Referring to Art. 3 we find that the total pressure on the vertical stem

$$P_1=\tfrac{1}{2}(100)\ (15.5)^2\ (0.866)\ \frac{0.866-0.281}{0.866+0.281},$$

$$=5310 \text{ lb.}$$

$$H_1=P_1 \cos 30^\circ=4600 \text{ lb.}$$

Thus the bending moment on the vertical wall at the top of the footing

$$M=(4600)\ (5.17)=23{,}780 \text{ ft.-lb.}$$

From Diagram 1 of Volume I for $f_c=500$ and $f_s=16{,}000$, we obtain $K=71$. Then

$$d=\sqrt{\frac{23{,}780}{71}}=18.3 \text{ in., say } 18\tfrac{1}{2} \text{ in.}$$

total depth 20 in. The effective depth may, with sufficient accuracy, be taken equal to the horizontal depth to steel. Diagram 1 shows $p=0.0050$; hence, area of steel $a_s=(0.0050)(12)(18.5)=1.11$ sq. in. per linear foot of wall. Diagram 3 shows $\frac{3}{4}$-in. round rods spaced $4\frac{1}{2}$ in. on centers will give an area of 1.18 sq. in. The wall will be tapered to 12 in. at the top.

At a point 10 ft. below the top of wall the bending moment is 88,400 in.-lb. and at 5 ft. below it is 11,000 in.-lb. The values of K which correspond are $\frac{88,400}{(12)(17.16)^2}=25$, and $\frac{11,000}{(12)(14.58)^2}=4.3$. Diagram 1 cannot be used to find the corresponding values of p, but, using 0.90 as a rough value for j and substituting in the formula $p=\frac{K}{f_s j}$, we have the values 0.0017 and 0.0003 respectively. The actual values of p, assuming one-third of the

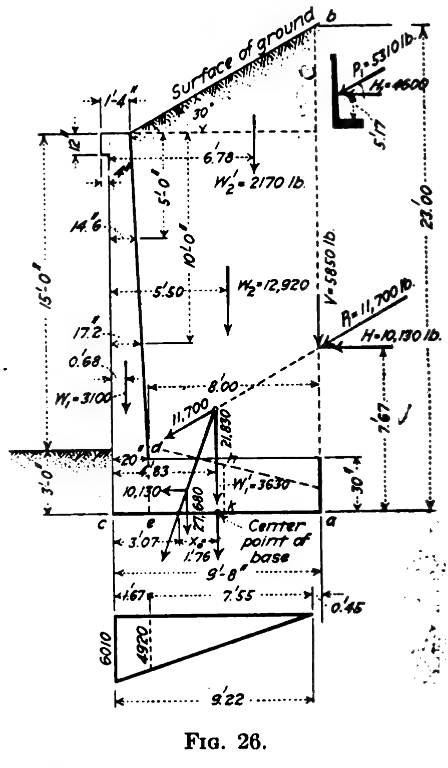


Fig. 26.

total steel at the 10-ft. point and one-sixth at the 5-ft. point, are one-third of 0.0050=0.0017 and one-sixth of 0.0050=0.0008. Thus if every third rod should be carried up to within 5 ft. of the top and one out of every six to the top of the wall, the design will be safe.

The shear on the vertical stem is due to the horizontal component of the earth pressure P_1. The shear is a maximum at

the top of the footing, and equals 4600 lb. The corresponding unit shear ($j=0.893$, Diagram 2)

$$v=\frac{4600}{(12)(0.893)(18.5)}=23 \text{ lb. per square inch}$$

which may be easily carried by the concrete.

The bond stress

$$u=\frac{4600}{\left(\frac{12}{4.5}\right)(2.36)(0.893)(18.5)}=44 \text{ lb. per square inch}$$

and is within the allowable.

Each vertical rod should either extend at least $50\times\frac{3}{4}=37\frac{1}{2}$ in. into the footing or be securely anchored. Approximately the same percentage of shrinkage and temperature reinforcement will be employed in the vertical stem as in Design No. 1.

Footing.—The resultant pressure on the base of wall and the point of application of this pressure will need to be determined. Referring to Fig. 26,

$$W_1=\frac{1.0+1.67}{2}(15.5)(150)=3100 \text{ lb.}$$

$$\text{lever arm}=\frac{(1.0)^2+(1.0)(1.67)+(1.67)^2}{3(1.0+1.67)}=0.68 \text{ ft.}$$

$$W'_1=(2.5)\ (9.67)\ (150)=3630 \text{ lb.}$$

$$\text{lever arm}=4.83 \text{ ft.}$$

$$W_2=\frac{8.67+8.00}{2}(15.5)(100)=12{,}920 \text{ lb.}$$

$$\text{lever arm}=9.67-\frac{(8.00)^2+(8.00)(8.67)+(8.67)^2}{3(8.00+8.67)}=5.50 \text{ ft.}$$

$$W'_2=(8.67)\ \tfrac{1}{2}(8.67 \tan 30°)\ (100)=2170 \text{ lb.}$$

$$\text{lever arm}=1.00+\tfrac{2}{3}(8.67)=6.78 \text{ ft.}$$

The weight of the coping will be neglected.

$$r=\frac{(3100)(0.68)+(3630)(4.83)+(12{,}920)(5.50)+(2170)(6.78)}{3110+3630+12{,}920+2170}$$

$$=\frac{105{,}421}{21{,}830}=4.83 \text{ ft.}$$

and total weight is 21,830 lb.

The total earth pressure on the plane ab should now be computed, or

$$P=\tfrac{1}{2}(100)(23.00)^2(0.866)\left(\frac{0.585}{1.147}\right)=11{,}700 \text{ lb.}$$

The line of pressure drawn for P and the resultant weight intersects the base 3.07 ft. from the toe c.

The factor of safety against overturning is greater than 4. The factor of safety against sliding is $\frac{27,680(0.4)}{10,130}=1.1$. The tendency of the wall to slide is 10,130 lb., while the frictional resistance to sliding is $27,680\times 0.40=11,070$ lb. It would probably be wise to construct a 12-in. projection on the underside of the footing, as shown in Plate II, to increase the bearing against the soil in front of the wall. This projection may be placed in the middle of the base or at either end.

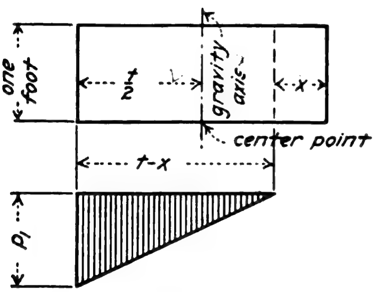


FIG. 27.

It should be noticed that the resultant pressure on the base strikes just outside the middle third. This, it should be understood, is not serious in a reinforced-concrete wall when the earth-pressure theory is used. However, for this case formulas (1) and (2) of Art. 7 cannot be employed to give the intensity of pressure on the soil beneath the footing. Since the resultant does not cut the middle third, only a part of the base is under pressure. There is what might be considered a neutral axis, as shown in Fig. 27. Two equations may be determined. One equation results from equating the total pressure on the foundation to W, and the second from equating the moment of that pressure about the gravity axis to Wx_o. From these equations we obtain

$$x=3x_o-\frac{t}{2} \text{ and } p_1=\frac{2}{3}\cdot\frac{W}{\left(\frac{t}{2}-x_o\right)}$$

In the problem at hand $x_o=4.83-3.07=1.76$ ft., $t=9$ ft. 8 in. and $W=27,680$ lb. Substituting,

$$x=3(1.76)-4.83=0.45 \text{ ft. and } p_1=\frac{2}{3}\cdot\frac{27,680}{(4.83-1.76)}=6010 \text{ lb.}$$

This value of p_1 is practically the allowable soil pressure. Very often in a design of this kind it is more practicable to rest the wall upon piles than try to increase the length of footing so as to bring the maximum soil pressure to a safe value.

Another method of finding the maximum soil pressure when the resultant strikes outside the middle-third point is as follows: Determine the distance from the toe of the wall to the point where the resultant cuts the base. This distance in the case at hand is 3.07 ft. Now since the center of gravity of the pressure triangle (shown in Fig. 26) is vertically below the point where the resultant intersects the base, the length of the base under pressure is three times the length determined above—in this

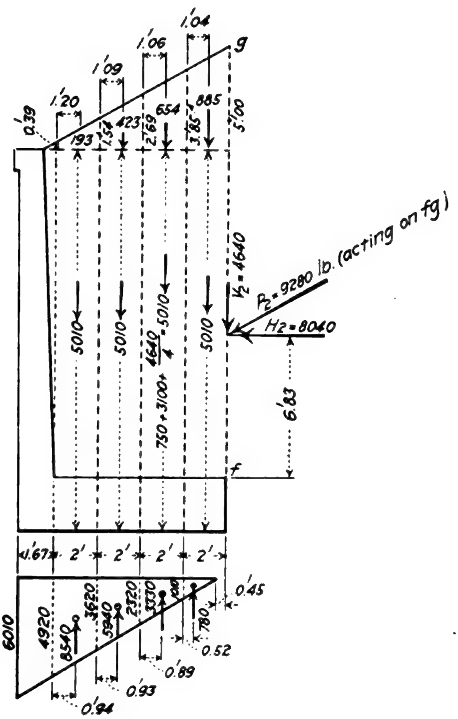


FIG. 28.

case 9.21 ft. The average pressure may now be found by dividing the total load by the length of effective base, and this multiplied by 2 gives the maximum pressure, which occurs at the toe of the wall. In the problem at hand $p_1 = \frac{(27,680)\ (2)}{(9.21)} =$ 6010 lb.

The second method given is much simpler than the first, but the first method is general and shows the manner of solving for

pressures on foundations where the part of the base under pressure is not of a rectangular shape.

When comparison is made with a plain concrete wall by the equivalent fluid pressure method, the resultant pressure should strike the base within the middle third, otherwise the reinforced wall will not have equal stability to the plain wall. This is not necessary, however, when the earth pressure theory is used; provided, of course, that the pressure on the soil is not greater than the allowable and provided that the wall has sufficient stability against overturning and against sliding.

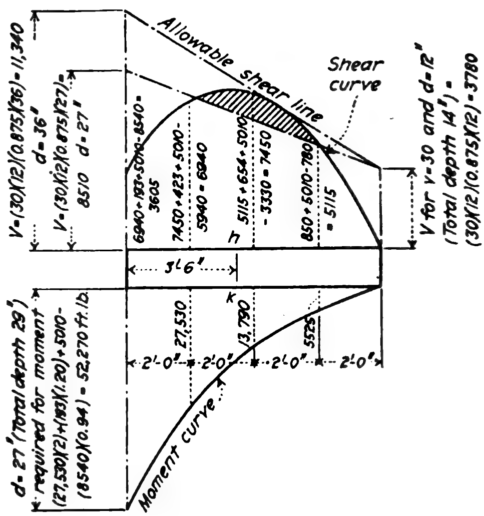
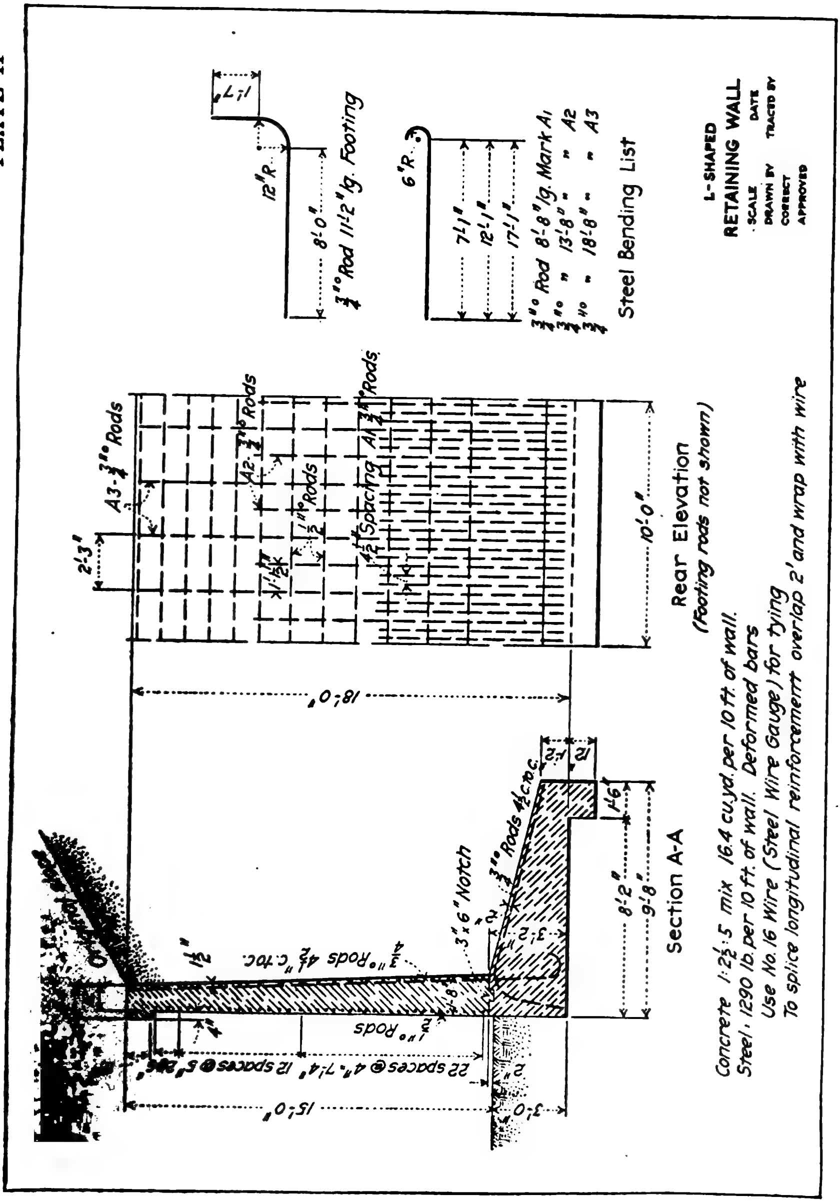


Fig. 29.

Footing Cantilever.—Fig. 26 shows the length of this cantilever to be 8.00 ft. The forces acting on this part of the footing are the upward pressure of the soil, the downward weight of the earth filling, and the vertical component of the earth pressure.

Fig. 29 gives the diagrams for shear and moment which may be easily obtained if a figure is first constiucted similar to Fig. 28. In Fig. 28 the quantities shown are the amounts of the forces represented by the respective areas. The arrangement is such that the foices to consider in finding shear and moment at any given section may be conveniently determined. The vertical component of the earth pressure to be taken into consid-

PLATE II


L-SHAPED
RETAINING WALL
SCALE
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DRAWN BY
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CORRECT
APPROVED
Section A-A
Rear Elevation
(Footing rods not shown)
Steel Bending List
Concrete 1:2½:5 mix 16.4 cu.yd. per 10 ft. of wall.
Steel. 1290 lb. per 10 ft. of wall. Deformed bars
Use No. 16 Wire (Steel Wire Gauge) for tying
To splice longitudinal reinforcement overlap 2' and wrap with wire
3"x6" Notch
18'-0"
15'-0"
10'-0"
8'-2"
9'-8"
3'-0"
2'-3"
12"R.
6"R.
8'-0"
1'-7"
7'-1"
12'-1"
17'-1"

eration is the vertical component of the pressure P_2 acting upon the plane *fg*. The value of V_2 is shown at the right of Fig. 28 for convenience.

It is not necessary for the student to construct the moment diagram, since it is evident from Fig. 29 that the footing is always sufficiently strong as regards moment if it is tapered from the required depth at the section *de*, Fig. 26, to zero at the inner end. However, this cannot be said in regard to the shear diagram since it is important that the variation of the shear should be considered in every case. The shear is not a maximum at the section *de* when the cantilever is of any considerable length and much error could result if the maximum shear were not determined. Fig. 29 illustrates this.

The minimum depth to steel for moment is (Diagram 1)

$$\sqrt{\frac{(52{,}270)\ (12)}{(71)\ (12)}}=27 \text{ in.}$$

or a total depth of 29 in.

$a_s=(0.0050)\ (12)\ (27)=1.62$ sq. in. per linear foot of wall. The footing, however, will need to be deepened at the vertical stem to a total depth of 38 in. ($d=36$ in.) to provide for shear. The effective depth may, with sufficient accuracy, be taken equal to the vertical depth to steel.

Then

$$K=\frac{(52{,}270)\ (12)}{(12)\ (36)^2}=40.3$$

and Diagram 1 gives $p=0.0027$.

$a_s=(0027)\ (12)\ (36)=1.17$ sq. in. per linear foot of wall. Thus the same size and spacing of rods may be employed as in the vertical stem. Fig. 29 shows that the shear is satisfactory if a uniform taper is given to the footing to a depth of 14 in. (12 in. to steel) at the end. (See Plate II.)

The rods in the footing should extend at least $37\frac{1}{2}$ in. to the left of the point *d*, Fig. 26, in order to obtain sufficient grip on the concrete. To the right of *d*, it is proposed to stop off every other rod at the half-way point; namely, at *h*, Fig. 26. This arrangement would give sufficient grip for all rods in the footing. Maximum bond stress, however, should be investigated before the design is accepted.

The critical bond stress will occur at the section *hk*, Figs. 26

and 29, since at this section the shear is a maximum and only every other rod can be taken into consideration.

$$u = \frac{7500}{\left(\frac{12}{9.0}\right)(2.36)\left(\frac{7}{8}\right)(25.5)} = 107 \text{ lb. per square inch.}$$

This stress is above the allowable and all the rods must be carried through to the end of footing.

The complete design is shown in Plate II.

12. Design of Counterforted Type.—In a counterforted type of wall the vertical slab is supported at intervals by vertical ribs or counterforts. These counterforts act as cantilevers and are securely tied to both the vertical wall and the footing. The projecting toe of the footing is a cantilever while the inner portion is a slab supported by the counterforts.

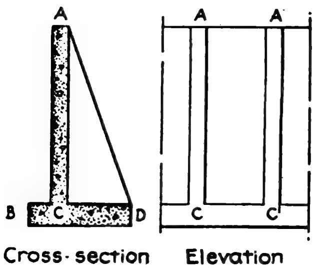


Fig. 30.

Since the cost of constructing the forms for this type of wall is greater than that for the cantilever type, results show the counterforted wall to be the more economical when the height is greater than about 20 ft. The economical spacing of counterforts should be determined for each case. In the designs to follow, this spacing will *arbitrarily* be made 8 ft.

In designing, the curtain wall AC, Fig. 30, is usually considered as made up of a series of horizontal strips and treated as slabs partly continuous and uniformly loaded. The pressure against this wall changes with the height so that the pressure upon the different strips increases with the depth.

The footing slab marked CD, Fig. 30, may also be considered as composed of narrow strips uniformly loaded. The loading consists of the dead weight of the slab, the downward weight of the earth, and the upward reaction of the soil.

The projecting toe of the footing—namely, BC—should be treated in the same manner as the corresponding part in the cantilever type of wall. It is sometimes more economical when the projection is large to introduce small buttresses and construct also this part of the footing as a partly continuous slab.

The load on the counterforts comes from the curtain wall

which in turn takes the earth pressure. The thickness of counterforts must be sufficient to insure rigidity and give necessary space for the reinforcing rods.

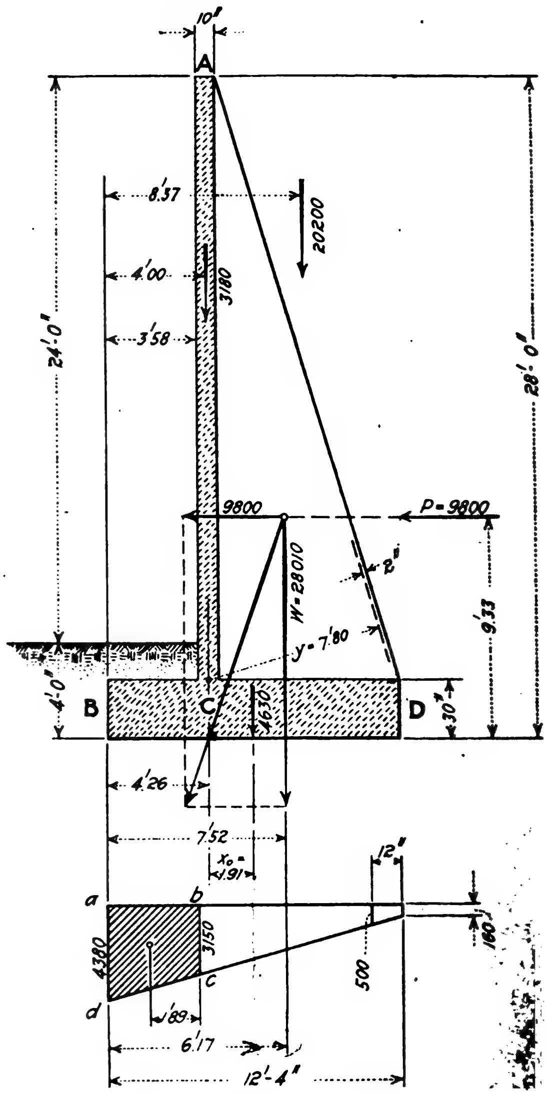


FIG. 31.

Design No. 1. Design a reinforced-concrete retaining wall with counterforts to support a level bank of earth and to be

24 ft. high above the ground. Maximum allowable pressure on the base is 5000 lb., or 2½ short tons per square foot. Coefficient of friction of concrete on the earth foundation is 0.60. The weight of earth and concrete, and the working stresses for the concrete, will be taken the same as in the preceding design. Deformed rods will also be assumed. The equivalent fluid weight will be taken at 25 lb. per cubic foot. Counterforts will be made 16 in. in thickness and spaced arbitrarily 8 ft. on centers.

The base of wall will be placed 4 ft. below the ground level, making $h=24+4=28$ ft. The trial value of $\frac{t}{h}$ from the table of Art. 9 is 0.44, so that $t=12$ ft. 4 in. BC, Fig. 31, will arbitrarily be made 4 ft. The tables of Volume I will be used in the design.

Vertical Wall.—The thickness of the footing will be assumed at 30 in. Then the height of the curtain wall above the top of the footings will be 25½ ft. The pressure at a depth of 25½ ft. is $wh=(25)\ (25.5)=638$ lb. per square foot. Considering a horizontal strip of the curtain wall at this depth and employing the formula $M=\frac{wl^2}{10}$ for a partly continuous slab,

$$M=\frac{(638)\ (8)^2}{(10)}=4080 \text{ ft.-lb. per foot of width.}$$

Then

$$d=\sqrt{\frac{4080}{71.3}}=7.6 \text{ in., say 8 in. (total thickness 10 in.)}$$

and

$$a_s=(0.0050)(12)(7.6)=0.46 \text{ sq. in. per foot of width.}$$

Round rods ½ in. in diameter and spaced 5 in. center to center will be sufficient. In any design, the vertical wall should have a thickness of at least 8 in., as it is difficult to properly place concrete where the wall is very thin.

The bending moment at the counterforts is negative and numerically equal to the positive moment at the middle of the span. The moment is negative for about one-fifth of the span on each side of the counterforts and may be provided for by introducing short rods or by bending the main slab rods to the inner edge at these points. The first method will be adopted in this problem. The short rods should have a length on each

side of the counterforts of $50 \times \frac{1}{2} = 25$ in. for straight bond, and this length in this case is greater than one-fifth the span.

The thickness of the vertical wall as determined for moment must now be tested for bond and for shear. From the nature of the loading and the connection of the slab with the footing, and also from the fact that the maximum bond and maximum shear will occur only at the bottom of the lowest strip, the span of slab, as far as shear and bond is concerned, will be taken as the clear distance between counterforts. The same span might be considered for moment, but moment does not control. The total shear

$$V = (638)\ (4.00 - 0.67) = 2130 \text{ lb.}$$

Then, assuming the same size and spacing of rods over supports as at the center of span,

$$u = \frac{2130}{\left(\frac{12}{5}\right)(1.571)\ (0.894)\ (8)} = 79 \text{ lb. per square inch}$$

which is satisfactory. The unit shear

$$v = \frac{2130}{(12)\ (0.894)\ (8)} = 25 \text{ lb. per square inch}$$

which is allowable.

For ease in construction the thickness of the curtain wall will be made the same throughout and the spacing of the horizontal rods will be increased with the decreasing pressure toward the top, as shown in Plate III. As the temperature stresses increase toward the top, the spacing of the rods for the upper one-half of the height will be kept uniform. At a point one-fourth the height of wall above the footing, the required spacing is $5 \times \frac{4}{3} = 6.7$ in., and at one-half the height it is $5 \times 2 = 10$ in.

The wall is fully reinforced for shrinkage and temperature stresses in a horizontal direction by the main reinforcement. Round rods $\frac{1}{2}$ in. in diameter and spaced 24 in. center to center will be placed vertically to take the temperature stresses in that direction and also to serve as stress distributors.

Footing.—By proceeding as in the previous designs, the resultant weight is found to be 28,010 lb. for 1-ft. length of wall, and acts at a distance 7.52 ft. from the toe *B*. The difference in weight between a counterfort and that of an equal volume of earth backing is neglected, as this amounts to very little pressure on the base assuming a reasonable distribution along the

length of the wall. The resultant pressure drawn for P and W cuts the base just inside the middle third point. The distance from the toe B to this intersection scales 4.26 ft., or analytically, $\frac{(9.33)\ (9800)}{28010}=4.26$ ft. Thus the computations show a slightly greater safety than the necessary conditions for our design. The thickness and shape of the footing, however, have not yet been definitely determined.

The intensity of pressure on the soil at each end of the base may be found by using the formulas

$$p_1=\frac{W}{t}\left(1+\frac{6x_0}{t}\right)$$

$$p_2=\frac{W}{t}\left(1-\frac{6x_0}{t}\right)$$

of Art. 9. In this design

$$x_0=1.91 \text{ ft., } W=28{,}010 \text{ lb., and } t=12.33 \text{ ft.}$$

Substituting,

$$p_1=\frac{28{,}010}{12.33}\left(1+\frac{6\times1.91}{12.33}\right)=4380 \text{ lb. per square foot.}$$

$$p_2=\frac{28{,}010}{12.33}\left(1-\frac{6\times1.91}{12.33}\right)=160 \text{ lb. per square foot.}$$

Fig. 31 shows the distribution of pressure throughout the length of the base.

The wall is safe against sliding, having a factor of safety of $\frac{(28{,}010)(0.6)}{9800}=1.7$. The factor of safety against overturning is $\frac{7.52}{7.52-4.26}=2.3$.

Inner Floor Slab.—The loading on the horizontal footing slab *CD*, Fig. 31, is the difference between the downward forces and the upward pressure of the soil. This difference is a maximum at D and decreases toward C, so a strip of footing at D 1 ft. wide will be considered.

The upward pressure is $\frac{160+500}{2}=330$ lb. and may be considered as uniformly distributed, while the weights of the earth and footing are 2550 lb. and 375 lb. respectively. Thus the uniform load on an end strip 12 in. wide is 2550+375−330= 2595 lb. per linear foot. The maximum bending moment in this slab (considering it as partly continuous)

$$M=\frac{(2595)(64)}{10}=16{,}600 \text{ ft.-lb.}$$

Then $$d=\sqrt{\frac{16{,}600}{71.3}}=15\tfrac{1}{4} \text{ in. (total depth } 17\tfrac{1}{2} \text{ in.)}$$

The depth of the slab is generally controlled by shear. The total shear at the edge of the counterfort is (2595) (4.00−0.67) =8640 lb. and the necessary depth for shear, assuming $j=\frac{7}{8}$, is

$$d=\frac{8640}{(12)\left(\frac{7}{8}\right)(30)}=28 \text{ in. (total depth 30 in.)}$$

Thus the total thickness of the slab should be made 30 in. instead of $17\frac{1}{2}$ in. as required for moment. It seems reasonable here (as in the vertical wall) to consider the span of slab for shear and bond as the clear distance between counterforts. Only the end strip receives maximum stress and this is strengthened by the strips adjoining.

Negative bending moment will be provided for by separate short rods at the top of slab at counterforts. Since maximum bond stress will occur in these rods, it is the intention to design the bottom rods simply for the bending moment, giving a sufficient spacing to the top rods to take care of the bond stress.

The size and spacing of the bottom rods will now be determined for moment.

$$K=\frac{M}{bd^2}=\frac{16{,}600}{(28)^2}=21.2$$

Table 3 cannot be used to find the corresponding value of p, but using 0.93 as a rough value for j and substituting in the formula $p=\frac{K}{f_sj}$, we have

$$p=\frac{21.2}{(16{,}000)(0.93)}=0.0014$$

Thus $$a_s=(0.0014)(12)(28)=0.47 \text{ sq. in.}$$

and $\frac{3}{4}$-in. round rods spaced 11 in. on centers is sufficient for moment in the bottom rods

Rods of the same size will be used over supports. Bond will control.

$$\text{Spacing}=\frac{(80)(12)(2.36)(0.93)(28)}{8640}=6.8 \text{ in.}$$

A 6-in. spacing at the edge of footing will be considered satisfactory.

The thickness of the slab will be made uniform and the spacing

of the rods will be increased as the load decreases toward the vertical stem. Since bond and moment vary directly with the load, the spacing for both top and bottom rods will vary in the same ratio. For each 1-ft. strip along the footing, the unit load is decreased by $\frac{4380-160}{12.33}=342$ lb. At 2 ft. from the end the unit load is 2250 lb., and the required spacing in the top rods is $\frac{(6.8)(2595)}{2250}=7.8$ in. and in the bottom rods $\frac{(11)(2595)}{2250}=12.7$ in.; at 4 ft. from the end the unit load is 1570 lb., and the required spacing in the top rods is 11.2 in. and in the bottom rods 18.2 in. As a general rule the spacing of the rods should not be increased much beyond 18 in. center to center.

The top rods should have a length on each side of the center of counterfort sufficient to prevent failure by bond.

$$p=\frac{0.44}{(6)(28)}=0.0026$$

$$j=0.919 \text{ (from Table 3)}$$

$$f_s=\frac{(16{,}600)(12)}{(0.0026)(0.919)(12)(28)^2}=8900 \text{ lb. per square inch}$$

$$\frac{fi}{4u}=\frac{(8900)(\frac{3}{4})}{(4)(80)}=21 \text{ in.}$$

Thus 21 in. on each side of counterfort is sufficient. This length is practically one-fifth the span.

Using the exact value for j, we have

$$v=\frac{8640}{(12)(0.919)(28)}=28 \text{ lb. per square inch}$$

which is satisfactory.

Outer Cantilever.—The computations for this part of the footing are similar to those for the corresponding part of the cantilever wall footing. Fig. 31 shows the length to be 3.58 ft. The downward weight is that of the cantilever itself and the upward pressure is represented by the trapezoid *abcd*. The distance of the center of gravity of this trapezoid from the line *bc* is

$$3.58-\frac{3.58}{3}\times\frac{2(3150)+4380}{3150+4380}=1.89 \text{ ft.}$$

The moment close to the vertical wall is

$$M=\left(\frac{4380+3150}{2}\right)(3.58)(1.89)-(3.58)(2.5)(150)\left(\frac{3.58}{2}\right)$$
$$=23{,}100 \text{ ft.-lb.}$$

The minimum depth to steel for moment should be

$$\sqrt{\frac{23{,}100}{71.3}}=18.0 \text{ in.}$$

and the area of steel $a_s=(0.0050)(12)(18.0)=1.08$ sq. in. Round rods $\frac{3}{4}$ in. in diameter and spaced $4\frac{1}{2}$ in. on centers will suffice.

The total shear

$$V=\frac{4380+3150}{2}(3.58)-(3.58)(2.5)(150)=12{,}160 \text{ lb.}$$

Assuming $\frac{7}{8}$ for j, the depth to satisfy shearing stress

$$d=\frac{12{,}160}{(12)(30)\left(\frac{7}{8}\right)}=39 \text{ in. (total depth 41 in.)}$$

For this depth with the same sized rods and same spacing as required for moment, the approximate bond stress

$$u=\frac{12{,}160}{\left(\frac{12}{4.5}\right)(2.36)\left(\frac{7}{8}\right)(39)}=57 \text{ lb. per square inch.}$$

A 6-in. spacing will be tried. $p=\frac{0.442}{(6)(39)}=0.002$ approximately, hence $j=0.928$. Then

$$u=\frac{12{,}160}{\left(\frac{12}{6}\right)(2.36)(0.928)(39)}=72 \text{ lb. per square inch}$$

which is within the allowable. The unit shear is obviously less than the allowable since the value of j is greater than $\frac{7}{8}$. The thickness of the footing may be reduced to 12 in. at the outer end.

The stress in the steel

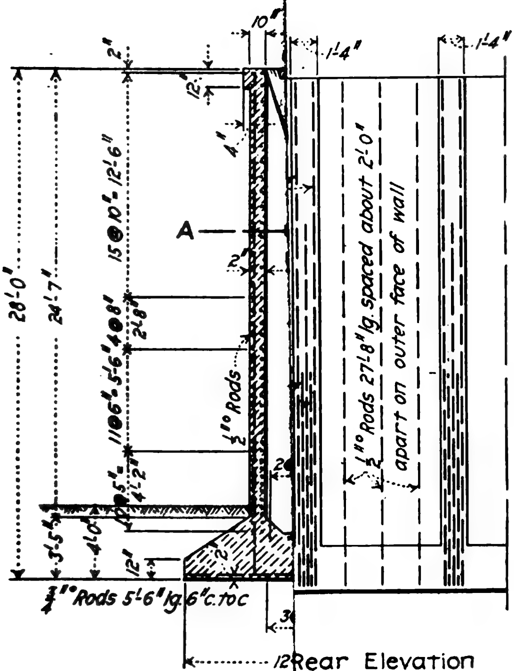
$$f_s=\frac{(23{,}100)(12)}{(0.0019)(0.928)(12)(39)^2}=8620 \text{ lb. per square inch}$$

and the length of embedment necessary to prevent slipping

$$\frac{fi}{4u}=\frac{(8620)(\frac{3}{4})}{(4)(80)}=21 \text{ in.}$$

The rods will have sufficient grip on the concrete if they are not formed into hooks in front, while more than 21 in. is available in the rear.

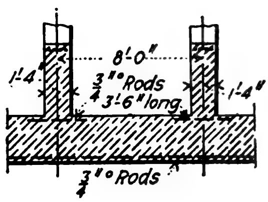
Counterfort.—The total force transmitted to a counterfort is,



Rear Elevation

Section C-C bet

Footing	7
	16
	12
Wall. *Horizontal*	41
	41
Vertical	2
	3
Counterfort	2
	1
	2
Vertical	2
	2
	2
	2
	2



Section B-B

Concrete 1:2½:5 mix
Deformed bars - Use No. 16 Wire (Steel Wire Gauge) for tying

COUNTERFORT RETAINING WALL

SCALE 1/4" = 1 FOOT OCT 16, 1912

DESIGNED BY TRACED BY

CHECKED BY APPROVED

(*Facing Page* 52.)

with sufficient accuracy, $\frac{1}{2}$ (25)(25.5)²(8) = 65,000 lb. Since the resultant force acts at one-third the height, the bending moment

$$M = (65,000)\left(\frac{25.5}{3}\right)(12) = 6,630,000 \text{ in.-lb.}$$

The tension in the counterfort along the edge AD, Fig. 31, will be carried by rods near this edge and the stress in these rods at any point may be found accurately enough by an equation of moments taken about the center of the front wall at the top of footing. Allowing 2 in. from center of steel to edge of counterfort, the moment arm y of the maximum stress in the rods scales 7.80 ft.

$$K = \frac{M}{bd^2} = \frac{6,630,000}{(16)(7.8)^2(12)^2} = 47.3$$

Table 3 shows $p = 0.0032$. Thus $a_s = (7.8)(12)(16)(0.0032) =$ 4.79 sq. in. and five $1\frac{1}{8}$-in. round rods will be needed. The thickness of the counterfort is sufficient to properly provide for these rods if a spacing of $2\frac{1}{2}$ diameters is allowed.[1] At 16 ft. below the top the bending is 1,638,400 in.-lb., and at 8 ft. below the top it is 204,800 in. The effective depths of the counterfort at these points are 5.00 ft. and 2.60 ft. respectively. The required values of p approximately corresponding are 0.002 and 0.001. If we consider 3 of the rods to be carried above the 16-ft. point, and 2 above the 8-ft. point, then $p = \frac{(0.994)(3)}{(16)(5.00)(12)}$ $= 0.0031$ and 0.0040 respectively. These values are greater than those required and the rods may be stopped off as planned.

In addition to the inclined bars placed near the edge of the counterfort, horizontal and vertical bars are necessary to tie the vertical and horizontal slabs to the counterfort, and to transfer the stresses. Since concrete is assumed to be incapable of taking tension, these rods must be considered to take all the shear along the horizontal and vertical edges of the counterfort. At the top of the footing the shear on 1 ft. in height is (25)(25.5)(6.67) = 4260 lb. If $\frac{1}{2}$-in. round rods are used, the required number in a foot of height is $\frac{4260}{(0.196)(16,000)} = 1.35$. These rods should be placed in pairs, each hooked around the outer horizontal reinforcing bars in order to obtain a satisfactory

[1] The Joint Committee in their second report recommends 3 diameters; see report at the end of this volume.

bond. The spacing of these pairs at the footing will be $\frac{12(2)}{1.35}$ $=17\frac{1}{2}$in. The spacing of the rods at other points in the vertical wall may be determined in a similar manner. The arrangement of the pairs of rods is shown in Plate III.

The spacing of the vertical bonding rods may be determined in a somewhat similar manner to the above. At *D*, Fig. 31, the downward tension on the counterfort caused by the shear from the floor slab on both sides of the counterfort is $(2595)(6.67)$ $=17{,}300$ lb. on 12 in. of length. On the succeeding 12-in. strips the tension is decreased by $\frac{4380-160}{12.33}(6.67)=2280$ lb. for every foot distant from *D*. The diagonal rods at the extremity of the footing will be considered to take the tension for only the first 12 in. For the second foot strip the total tension is 15,020 lb. and two ¾-in. round rods will be considered sufficient if placed at the center of this strip—namely, 1½ ft. from the end of footing. The tension coming from the third foot strip is 12,740 lb. and two ¾-in. round rods will also serve here if placed 2½ ft. from the end. Two ¾-in. round rods will serve the next or fourth strip, two ¾-in. round rods the fifth strip, and two ¾-in. round rods for the remaining length of 2.92 ft.

The vertical and diagonal rods of the counterfort should be extended 50 diameters into the footing or, if desired, the vertical rods may be hooked around the lower reinforcement.

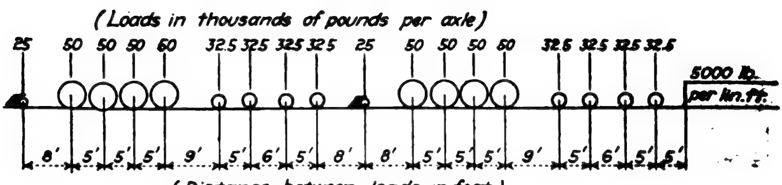


FIG. 32.

Design No. 2. Design a counterfort retaining wall to have a total height of 20 ft., and to carry a sand filling which weighs 100 lb. per cubic foot. The wall is also to carry a surcharge due to Cooper's E50 standard train loading (Fig. 32) on each track shown in Plate IV. The weight of the fluid to give the same pressure as that of the filling will be taken at 25 lb. per cubic foot. It will be assumed that the face of the retaining wall is to be placed

on the right-of-way line where it is not possible to extend the toe of the foundation beyond the face of the wall. A pressure of 6000 lb. per square foot is allowable on the soil, and the coefficient of friction of concrete on the earth foundation is 0.40. The counterforts will be spaced arbitrarily 8 ft. on centers. The

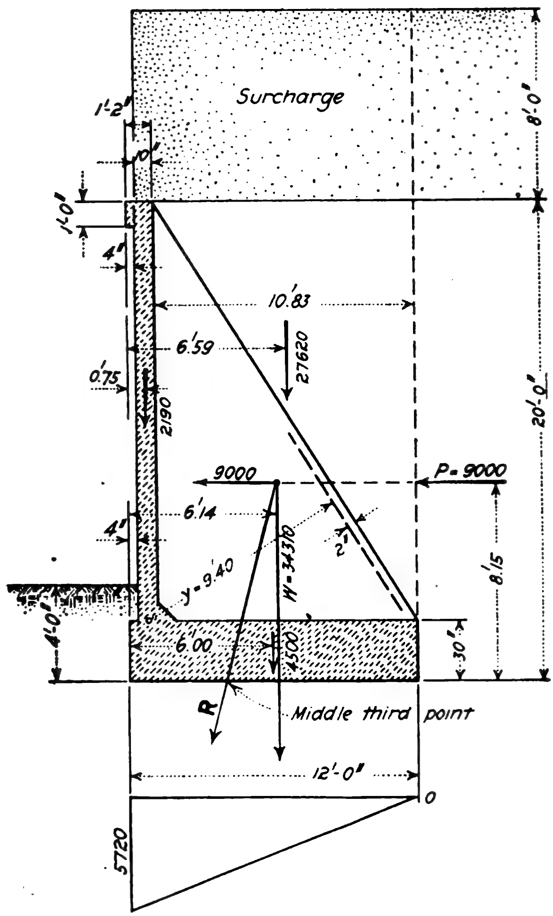


FIG. 33.

working stresses for the concrete will be taken the same as in the two preceding designs. Round deformed rods will be assumed as before.

A little investigation will show that maximum conditions will occur with a train load on each track and with the drivers

of the locomotives directly opposite each other. It will be assumed that each track load is distributed laterally over a width equal to the distance between track centers, namely, 13 ft. If we consider a distance of 10 ft. in the direction of the rails, the maximum load on each track may be taken 100,000 lb. occurring under the locomotive drivers. This load is thus assumed to act over an area of $10 \times 13 = 130$ sq. ft giving a weight per square foot of $\frac{100,000}{130} = 770$ lb., or about 800 lb., considering the weight of rails and ties. This is equivalent to a surcharge of 8 ft. and our problem, then, is to design a counterforted wall to sustain a sand filling and an 8 ft. surcharge of the same material, as shown in Fig. 33.

Vertical Wall.—The thickness of the footing will be assumed at 30 in., making the height of the curtain wall 17½ ft. The

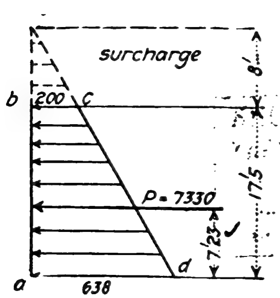


FIG. 34.

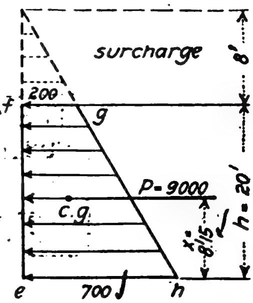
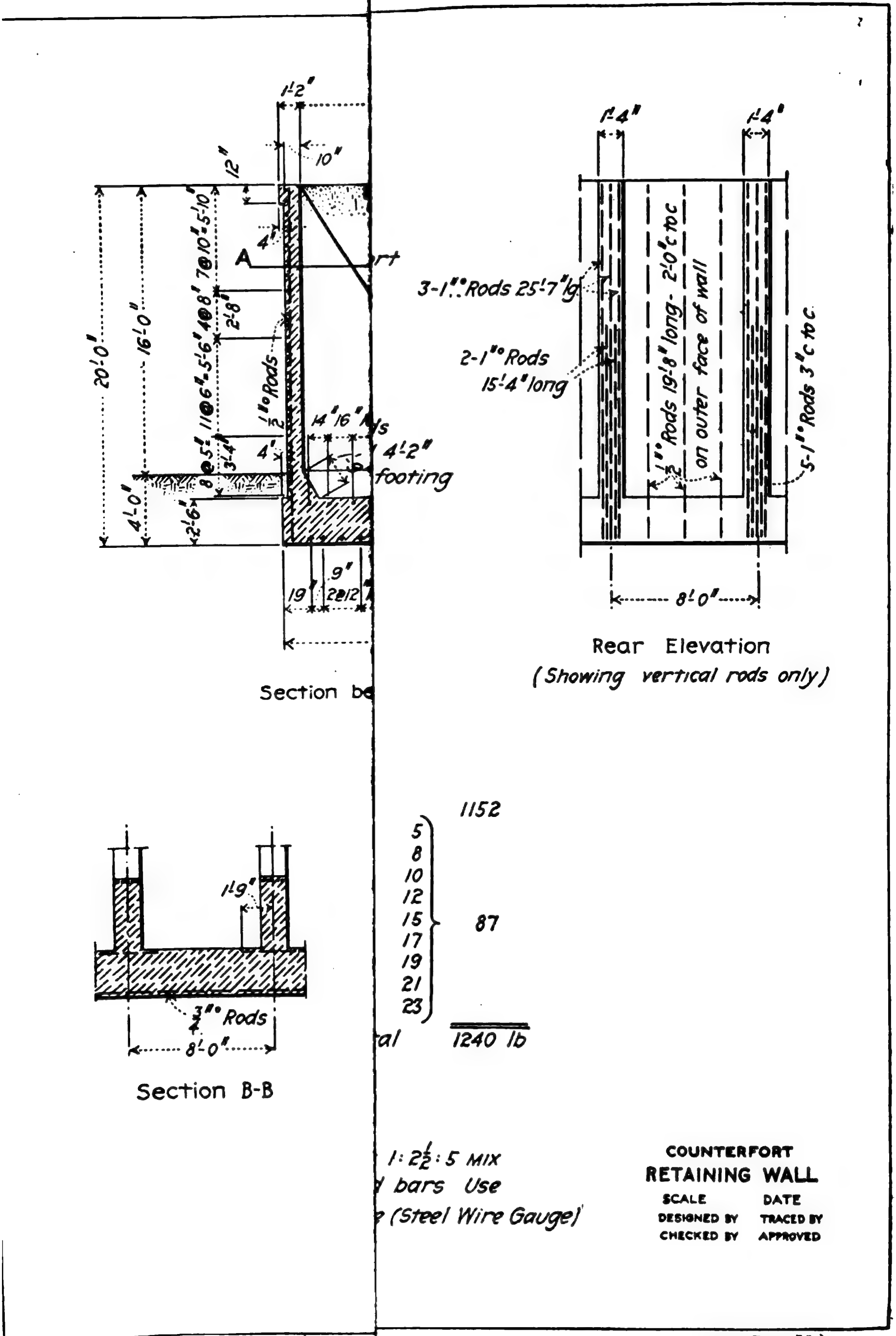


FIG. 35.

total force acting upon this slab is represented by the area *abcd* in Fig. 34. At any given depth the pressure is wh; that is, if h is taken as the total depth, including surcharge. For example, at the top of the footing the pressure per square foot $P = wh = (25)(25.5) = 638$ lb.; and, at the top of wall, $P = (25)(8) = 200$ lb.

The pressure at the bottom of the vertical slab is the same as in the preceding design and, since the counterforts have the same spacing in both designs, the thickness of slab and spacing of the rods at the bottom may be the same. At a point one-fourth the height of wall above the footing, the required spacing is $5 \times \frac{25.5}{21.1} = 6.0$ in., and at one-half the height it is $5 \times \frac{25.5}{16.75} = 7.6$ in.



(Facing Page 56.)

Footing.—The total force P acting upon the retaining wall is represented by the area *efgh*, Fig. 35. By referring to Art. 3 it is clear that $P=\frac{1}{2}\,wh\,(h+2h_1)=\frac{1}{2}(25)(20)(36)=9000$ lb. and acts at a distance above the base $x=\dfrac{h^2+3hh_1}{3(h+2h_1)}=\dfrac{(20)^2+3(20)(8)}{3(36)}$ $=8.15$ ft. The same distance x may be obtained by finding the distance from *eh* to the center of gravity of *efgh* by the formula employed in the preceding designs, or $x=\dfrac{20}{3}\cdot\dfrac{(2)(200)+700}{200+700}$ $=8.15$ ft.

The resultant weight is found to be 34,310 lb. for 1-ft. length of wall, and acts at a distance 6.14 ft. from the toe. The resultant pressure drawn for P and W cuts the base exactly at the middle third point.

The average unit pressure on the base is $\dfrac{34{,}310}{12}=2860$ lb., and the maximum pressure at the toe is twice this value, or 5720 lb., which is within the allowable. The pressure at the inner extremity of the footing is, of course, zero.

The wall is safe against overturning and sliding, the factors of safety being 2.8 and 1.5 respectively.

Floor Slab.—We have essentially the same forces acting on the foot strip at the inner extremity as in the preceding design. The load on this strip determines the thickness of slab and $d=28$ in. will be taken as before, with $\frac{3}{4}$-in. round rods and the same rod spacing. The uniform load on the end 1-ft. strip is $2550+375-\dfrac{5720}{(12)(2)}=2685$ lb. as against 2595 lb. of the preceding design.

The total shear at the edge of the counterfort is $(2685)(4.00-0.67)=8940$ lb. The resulting bond stress

$$u=\frac{8940}{\left(\frac{12}{6}\right)(2.36)(0.919)(28)}=74 \text{ lb. per square inch}$$

and

$$v=\frac{8940}{(12)(0.919)(28)}=29 \text{ lb. per square inch.}$$

These values are within the allowable.

For each foot strip toward the vertical wall, the unit load is decreased by $\dfrac{5720}{12}=477$ lb. The shear changes sign at 6.6 ft. from the inner edge of the footing. The long rods should be

placed in the top of slab from this point on and thus interchange with the short rods. The top and bottom rows of rods should overlap to some extent because of the approximations in retaining wall design.

Counterfort.—The total force transmitted to a counterfort is (7330)(8) = 58,640 lb. (see Fig. 34) and the bending moment

$$M = (58{,}640)(7.23)(12) = 5{,}090{,}000 \text{ in.-lb.}$$

Assuming a thickness of 16 in. and, allowing 2 in. from center of steel to edge of counterfort, the moment arm y of the maximum stress in the rods scales 9.40 ft.

$$K = \frac{M}{bd^2} = \frac{5{,}090{,}000}{(16)(9.4)^2(12)^2} = 25.0$$

Table 3 shows that a value of p somewhat less than 0.0020 may be used. Five 1-in. round rods will be sufficient.

The size and spacing of the horizontal and vertical bonding rods may be determined in the manner described in the preceding design.

Plate IV gives the complete design of this wall.

13. Special Types of Reinforced-concrete Walls.—Fig. 36 shows the cellular type of reinforced-concrete retaining wall. This wall is formed of two longitudinal curtain walls (A) and (B), connected by transverse walls (C). The vertical space between the parallel longitudinal walls and the transverse walls is filled with earth. This type of wall gives a lower maximum soil pressure than either the cantilever or counterforted types, but under average conditions its cost per linear foot is greater. Its use would seem to be restricted to poor soil with no opportunity to drive piles and where the wall must be built as close as possible to a given property line—conditions which often occur in railway work.

Fig. 37 shows a design of wall which gives a maximum soil pressure even less than the cellular type and which costs approximately the same per linear foot of wall. This design was employed by the Chicago, Milwaukee & St. Paul Ry. in a long retaining wall at Elgin, Ill., and was chosen in preference to the design shown in Fig. 36 on account of the much lower bearing pressure on the soil. The wall consists of a footing (Y) which supports a longitudinal wall (T) and the cross walls (U). The cross walls at the outer end support the girder (X), which spans

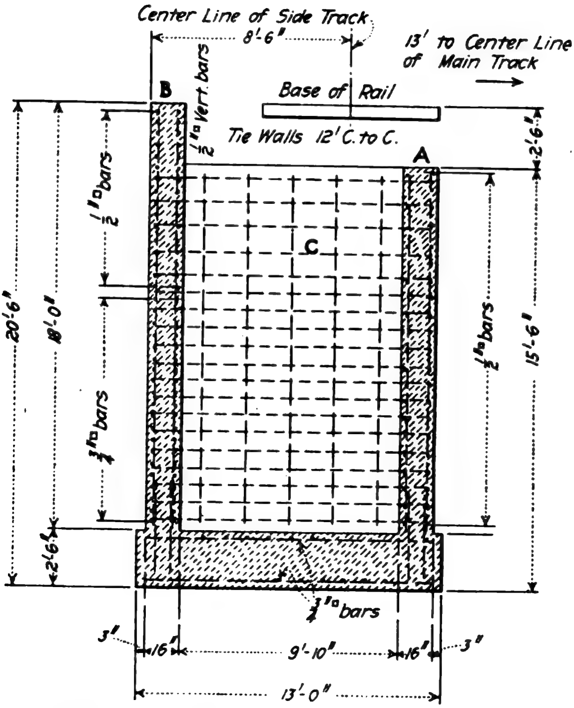

Center Line of Side Track
8'-6"
13' to Center Line of Main Track
Base of Rail
B
1/2" Vert. bars
Tie Walls 12' C. to C.
A
C
1/2" bars
3/4" bars
1/2" bars
20'-6"
18'-0"
2'-6"
15'-6"
2'-6"
3/4" bars
3"
16"
9'-10"
16"
3"
13'-0"

Fig. 36.

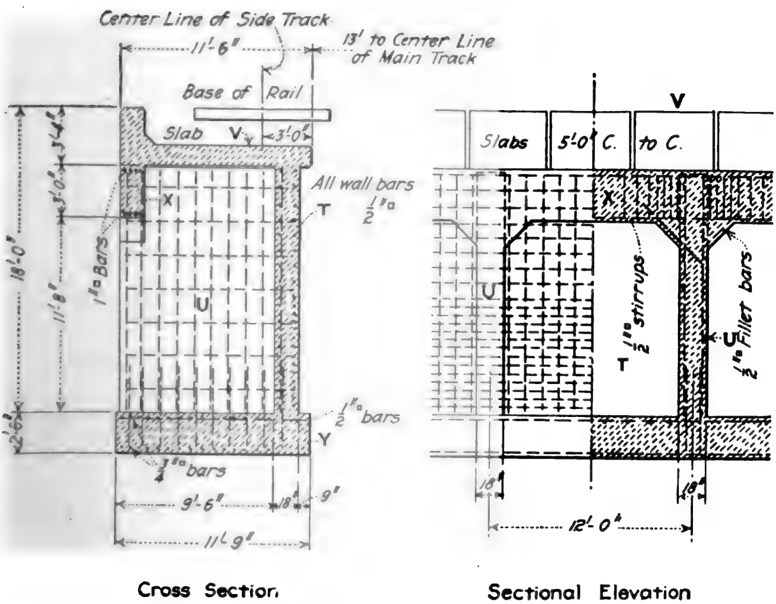

Center Line of Side Track
11'-6"
13' to Center Line of Main Track
Base of Rail
Slab
V
3'-0"
3'-4"
5'-0"
X
All wall bars 1/2"
T
1" Bars
18'-0"
11'-8"
U
2'-6"
1/2" bars
Y
3/4" bars
9'-6"
18"
9"
11'-9"
V
Slabs 5'-0" C. to C.
X
U
1/2" stirrups
T
U
1/2" Fillet bars
18"
18"
12'-0"
Cross Section
Sectional Elevation

Fig. 37.

from one cross wall to another. The slabs (V) are supported at one end by the girder and at the other end by the longitudinal wall. The great reduction in soil pressure is brought about mainly by eliminating the weight of the earth directly above the footings, the space between the cross walls (U) being an open and empty space in this design. In this type of wall, and in the cellular type as well, stability against sliding should be carefully investigated.

CHAPTER III

CONSTRUCTION

14. Back-filling and Drainage.—Quite as much attention should be paid to the earth filling and to its drainage as to the design and construction of the retaining wall proper or to the matter of providing a suitable foundation. If the earth is deposited in layers inclined from the wall, the pressure will be small compared to that resulting if the layers are sloped toward the wall. It is quite often the case that by depositing the back-filling so as not to slide against the wall, a light wall may be made to stand where, under the same conditions, if the earth is dumped so as to slide against the wall, even a heavy wall will fail.

When placing a back-fill near a steep undisturbed slope of earth or rock, care should be taken that the earth does not arch, or does not form a wedge. If the proper precautions are not taken, a heavy lateral thrust will occur near the top of wall.

Water behind a wall is a frequent cause of failure. It adds to the weight of the backing and softens the material so that the lateral thrust is increased. Also, undrained back-filling will freeze and create lateral thrust due to the consequent expansion. To drain the backing, *weepers* or *weep holes* should be left through the wall just above the footing. Tile, 3 or 4 in. in diameter is generally used and, in the North Central States, placed usually not more than 10 to 15 ft. apart. The tile should be connected with a longitudinal drain in front of the wall. If the backing is retentive of water, a vertical layer of broken stone, coarse gravel, or cinders should be placed next to the wall to act as a drain. The filling in front of the wall should also be carefully drained.

15. Forms.—Form work as applied to buildings will be treated in detail in Part II, Chapter XIX. Since the method of constructing forms and the directions for their removal are very much the same for different types of structures, very little need be said under this heading.

The lumber used for forms should have a nominal thickness of at least 1½ in. before surfacing and should be of a good quality of Douglas fir or Southern long-leaf yellow pine. The lumber for face work should be dressed on one side and on both edges to a uniform thickness and width. The lumber for backing and other rough work may be unsurfaced and of an inferior grade of the kinds above mentioned.

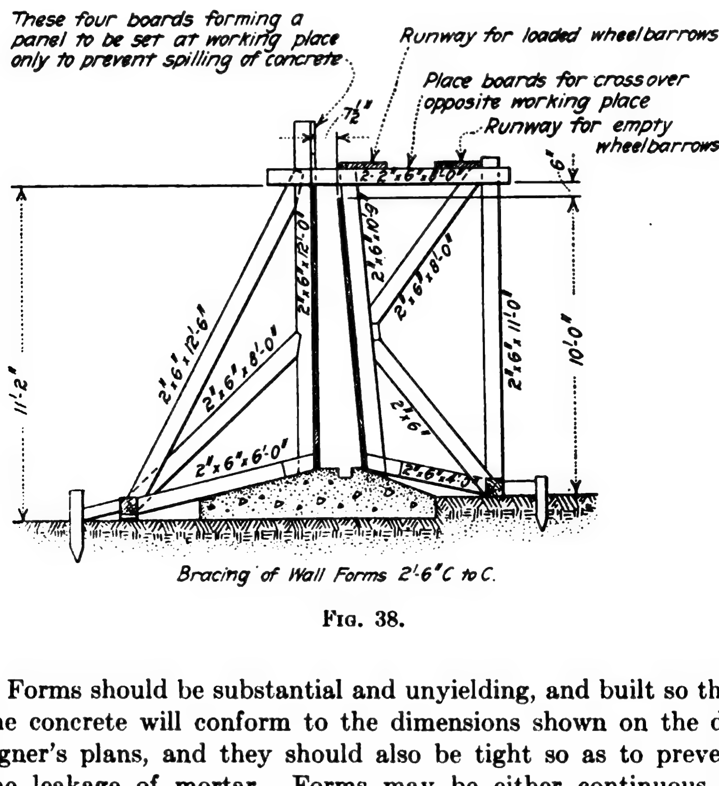


Fig. 38.

Forms should be substantial and unyielding, and built so that the concrete will conform to the dimensions shown on the designer's plans, and they should also be tight so as to prevent the leakage of mortar. Forms may be either continuous or sectional, or a combination of both, depending upon the economy of the work. The concrete in any given section should be allowed to harden for 36 hours before the forms are removed and, in freezing weather, extra care must be taken to make sure that the concrete has had sufficient time to become thoroughly set. Material once used for forms should be cleaned before being used again.

A design of a form for a cantilever wall is shown in Fig. 38.

This design was made by the engineers of the Isthmian Canal Commission, Pacific Division, for a reservoir wall and accompanies the Commission's annual report for the fiscal year ending June 30, 1910.

Courtesy of Atlas Portland Cement Co.

FIG. 39.—Reinforcement in place, Buffalo retaining wall.

Fig. 39 shows the method of constructing and reinforcing the counterforts of a retaining wall at Buffalo, New York. Fig. 40 shows the finished wall.

PROBLEMS

2. Design a cantilever retaining wall to be 18 ft. high above the ground and to support an earth bank whose surface has an upward $1\frac{1}{2}$ to 1 slope from the top of wall. The face of the wall is to be placed on a property line and the foundation is to be 4 ft. below the ground surface. Assume

the same weights for the earth and concrete as in the designs given for illustration, and take 3 short tons per square foot as the safe bearing power of the soil. The coefficient of friction of the concrete upon the earth foundation is 0.40. Assume plain round rods of medium steel. Employ the working stresses recommended by the Joint Committee for a 2000-lb. concrete and design by the equivalent fluid-pressure method, using 25 lb. per cubic foot as the equivalent fluid weight. Take a length of base which will bring the resultant pressure within the middle third of the base and assume the thickness of the footing at 30 in. Use tables of Vol. I.

Courtesy of Atlas Portland Cement Co.

FIG. 40.—Buffalo retaining wall, D. L. & W. R. R.

3. Design the wall of the preceding problem using Rankine's formula for earth pressure. Take the angle of internal friction of the earth filling at 45°.
4. Design a counterforted wall to be 30 ft. high above the ground and to support a level bank of earth. The wall is also to sustain an additional 5-ft. surcharge. Place the vertical slab so as to give a minimum amount of material in the wall. Space counterforts 8 ft. center to center. All other data is to be taken the same as in Problem 2.

PART II

BUILDINGS

INTRODUCTION

This treatise will deal principally with reinforced-concrete buildings of the skeleton type of construction; in other words, it will be assumed in the discusssions which follow (unless otherwise stated) that the student has in mind the usual type of concrete building (Fig. 41), in which there is an exterior as well as an interior frame of reinforced concrete and in which the walls

Courtesy of Turner Construction Co.

FIG. 41.—Model factory. Bush Terminal Co.'s buildings, Nos. 19 and 20. Total floor area 23 acres. This is the largest reinforced-concrete building constructed in the East.

(if any) are simply light curtain or panel walls of either concrete or brick, built at some convenient time after the adjacent framework is completed. Outside bearing walls with few openings are sometimes employed in reinforced-concrete construction, but it is in the former type of structure that the present-day builder

is particularly interested, because it is the easier to build and the more economical.

Sometimes only the basement walls of buildings are of the bearing type. In a common form of such construction, described in Art. 53, the basement wall between any two consecutive piers is made sufficiently deep to act as a beam and to distribute the load over the entire bay from column to column.

Courtesy of Turner Construction Co.

Fig. 42.—Candy factory. Wallace & Co., Brooklyn, N. Y. Note pleasing effect of combined brick and concrete finish, using brick curtain walls and brick veneer.

The skeleton type of construction is not always apparent in completed buildings (Fig. 42), especially at first glance, since, from an architectural standpoint, it sometimes becomes necessary to cover the exterior columns and walls with brick, terra-cotta, marble, limestone, or other material. The methods of tying a brick or stone facing to the concrete will receive attention in Art. 55.

Reinforced-concrete slabs are quite often employed in the floors and roofs of steel-frame buildings. The different ways of placing the concrete slab with reference to the steel frame and the manner of covering the beams and girders for fire protection will be described in Art. 20.

In order to obtain the fire-proof and sanitary features of concrete construction, it is quite common to employ an interior frame of reinforced concrete with the ordinary brick bearing-wall structures which are common to every community. The methods of design given in this text will apply to every part of this type of construction except the simple details of the brick walls.

SECTION I

DESIGN

CHAPTER IV

FLOORS

16. General Types.—There are four general types of reinforced-concrete floors: (1) monolithic beam and girder construction, (2) flat slab construction, (3) unit construction, and (4) steel frame construction with concrete slabs. In the fourth type mentioned, the beams and girders are usually covered with concrete for fire protection.

17. Monolithic Beam and Girder Construction.—The monolithic beam and girder floors may be divided into two groups—the first including solid concrete floors, and the second what is known as terra-cotta hollow-tile floors. The arrangement of beams and girders in the solid type has been considered in Art. 58 of Volume I, and much has been said of this type as to methods of design. This is by far the older of the two types, but the *hollow tile* is now used quite entensively for light buildings such as modern store buildings and office structures.

Fig. 43 shows a typical one-way hollow-tile slab and Fig. 44 a two-way tile construction. No cross-beams are employed in the one-way type except the small ribs of the floor slab formed between the rows of hollow tile. In the two-way type, cross-beams are placed at the columns. The tiles are placed directly upon the forms with the reinforcing rods in the spaces between them, and the concrete is filled in between the tiles and poured over the top to form the floor. The ribs form a series of comparatively light T-beams side by side with flanges usually two or more inches in thickness. The main beams or girders are also of T-shape. The flanges of these beams or girders are usually of the same thickness as the floor slab, but lighter tiles are sometimes used near the stem, in which case the flange becomes thinner than when the tiles are entirely omitted at this part of the floor. The function of the tiles is simply to create a void in the concrete and thus to decrease the dead weight of slab, and they do not enter into the calculations for strength of floor.

Either hard-burned or semi-porous tile may be used in reinforced-concrete floor construction. Hard-burned tile, due to its density, has a higher crushing strength and will, therefore, undergo a greater stress without any sign of failure, but it does not seem to be as good a fire-resisting material as the semiporous.

The following table gives average weights of the common sizes of hollow tile:

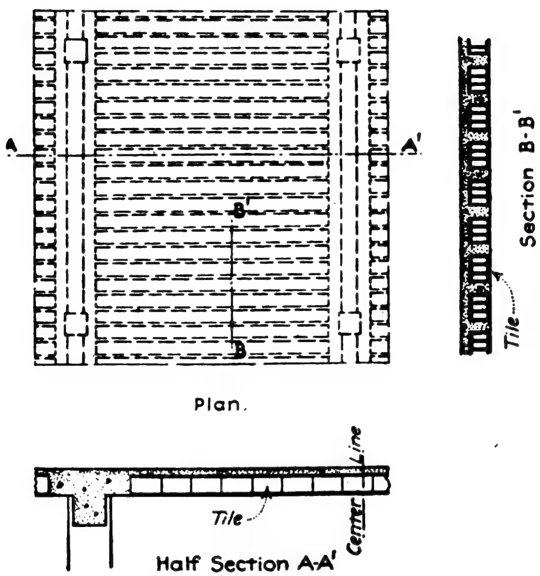


FIG. 43.

WEIGHTS OF HOLLOW TILE

4×12×12	16 lb.
5×12×12	20 lb.
6×12×12	22 lb.
7×12×12	27 lb.
8×12×12	30 lb.
9×12×12	33 lb.
10×12×12	35 lb.
12×12×12	40 lb.

The commercial sizes of tiles are usually 12 in. by 12 in. in plan and vary in depth from 4 in. to 16 in. The depth of a tile concrete floor should be designed so as to allow for these commercial sizes. The standard sizes manufactured by the

National Fire Proofing Company are all 12 in. by 12 in. in plan with the following depths: 4 in., 5 in., 6 in., 7 in., 8 in., 9 in., 10 in., 12 in., 15 in. Special sizes of tile may be obtained if the order is of sufficient size to warrant the manufacturing of the same.

Tiles are likely to vary $\frac{3}{8}$ in. from the dimensions specified so that the plans should show the full thickness of the floor and the minimum amount of concrete topping. If the tiles are small, due to shrinkage in burning, the thickness of floor should be made up in concrete.

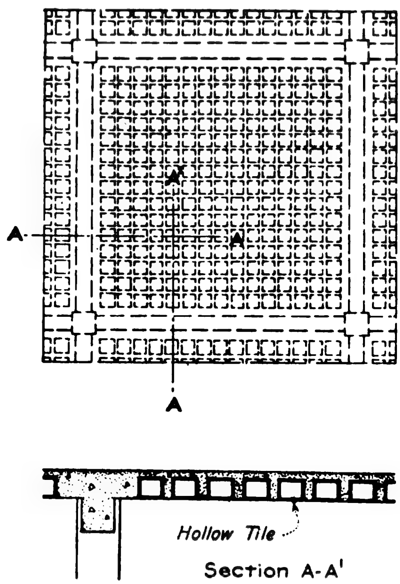


FIG. 44.

Unless the tile is thoroughly sprinkled before the floor is poured, slight depressions will occur over the ribs. This is because the hollow tile absorbs the moisture in the concrete of the top coat, causing it to set more quickly than the rib with its greater body of concrete and greater shrinkage. Sprinkling of the tile should be insisted upon, especially in hot weather.

Hollow-tile floors are generally plastered on the underside as it is only in the roughest kind of work that this is not done. The surface of the tiles should be deeply scored so that the plaster will bind firmly.

Ordinary hollow tile is open at both ends and cannot be used

when the floor is reinforced in both directions. For such floors two-way tile should be procured.

Hollow-tile floors are gaining in favor for long span construction, and where the loads are light and distributed. The reason for this is the small dead load, the flat ceiling, and the simplicity of the form work. The cost of the solid type of beam and girder construction for the conditions of long span and light loads is generally much greater than for hollow tile.

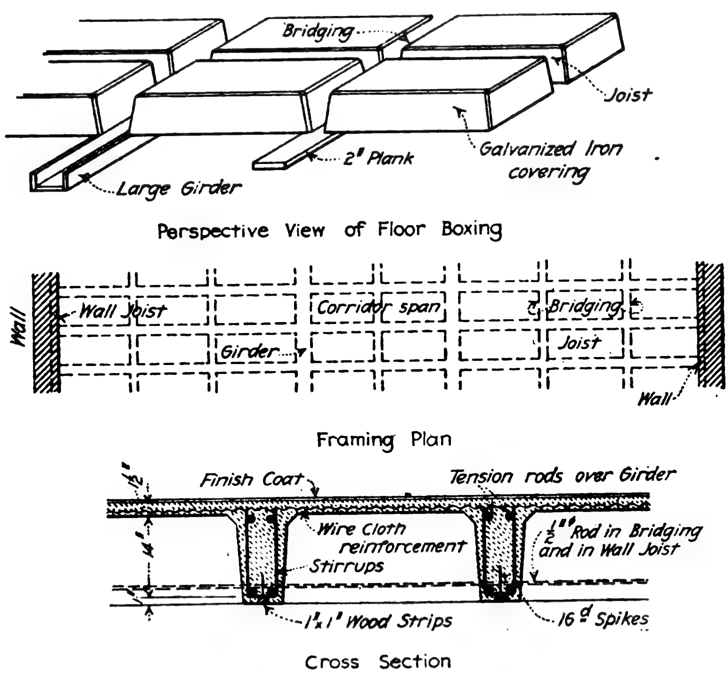


FIG. 45.—System of floor construction used at The University of Wisconsin.

A system of floor construction which compares favorably with the hollow tile, if not superior to it under certain conditions, is that used at The University of Wisconsin (Fig. 45). For the purpose of forming ribs, inverted wooden boxes covered with sheet metal are employed (instead of the usual forms) and these boxes can generally be used thirty times or more. The boxes which have slightly flaring sides and ends are laid face down upon a system of supporting planks upheld by shores and the

intervals between these boxes or cells, when filled with concrete, constitute the beams and cross-braces of the floor.

The following is quoted from a paper read by Mr. Arthur Peabody, Supervising Architect of The University of Wisconsin, at the third annual convention of the Engineering Society of Wisconsin:

"Other systems approach the practical value of this one in some degree, as for instance that one where hollow tiles are used in place of the wood cells. Where but one building is to be constructed, the cost of the cells might excel the cost of the tile. Where the building is of several stories, or in buildings of large enough area so that one part can be constructed before another, thus permitting the repeated use of a reasonable number of cells, the cost of the tile would probably exceed the proportionate cost of wood cells. The tiles serve no useful purpose except as forms between which the beams are cast, and their weight adds to the dead load of the floor construction.

"There is room for trouble in constructions where a certain beam supports a considerable area. Such a beam is usually calculated to do the work intended, with a good margin of safety, but suppose the casting of the beam proves to be faulty, or in some manner the beam suffers slight damage. At once the entire area is threatened. This suggests the arrangement of many beams of small size, each supporting a small floor space. To this there arises the objection of increased cost. Beams of long span and narrow width suggest cross bracing, and again the comparison with wood construction suffers as regards expense. The good elements of wood construction are, however, precisely those of advantage in concrete. Wood beams are of long span, slight dimensions, and are set closely together. Each beam is braced to its neighbor by cross bridging. Under the beams all sorts of pipings are extended, and a flat ceiling is placed beneath. Upon the beams the floor is laid. What could be more reasonable and convenient? Should a pipe fail it requires only the removal of the ceiling for its repair. Should it be necessary to change the dimensions of rooms, the necessary readjustments are readily made. Should a beam be slightly defective, or suffer injury, the beams adjacent will support the load, or if necessary, a section can be replaced without removing a large area of the floor. None of these advantages are to be had in the *factory type* of reinforced-concrete floor construction. (By the *factory type*, Mr. Peabody means the ordinary type of beam-and-girder construction dealt with in Volume I.)

"The system employed at the University of Wisconsin has been found practical, economical, and effective, and for buildings intended for ordinary use more satisfactory than the factory type of construction. In competition with other systems it has been found less expen-

sive where the requirements of the specification for the steel and other elements forming a necessary part of good construction have been conserved. This has been put to proof by obtaining alternate propositions on the same work, imposing only the requirements that the steel shall be strained not to exceed a certain amount, that spaces shall be provided for pipes, etc., and that the ceilings shall be flat.

"By the system described, spans from 12 to 26 ft. have been successfully cast at the University during the past two years. It is proposed to cast beams of 28-ft. span this year. These will require joists 8 in. wide on the bottom, 14 in. deep and 10 in. wide on top. With a spacing of 3 ft. 4 in. from center to center of joists the amount of concrete employed does not seem excessive. The cross braces in the floor, due to the intervals between the lengths of cells, have not been considered in the estimated strength of the floor. Their value in holding the joints against lateral strain is unquestionable and they serve also to distribute the load upon the floor. The increased width of the beams at the top, due to the flaring sides of the molds gives increased resistance to crushing, while the narrow bottom, enclosing the steel, is sufficient and economical.

"For attaching the ordinary ceilings to this construction small strips of wood are cast into the bottom of each joist and secured by bent nails, thus affording a ready means of applying the furring strips for the ceiling. These in turn give the needed spaces for pipes, etc. Upon the furring strips wire lath or plaster board is nailed and the plastering is applied.

"The most convenient size of cells appears to be 2 ft. 8 in. wide × 6 ft. 6 in. long. Beyond this size they cannot be so easily handled. Some cells were made 8 ft. long and a few 5 ft. At times it is necessary to build special cells to finish out a span.

"Variation in strength of floors was made by spacing the cells farther apart, making the concrete joists wider for the heavier floor. The percentage of steel was then increased according to rule. The joists were sometimes left exposed over laboratories, and made a very presentable appearance.

"The floor sheet, 2½ in. thick including the sand finish, seems fragile. Experience, however, shows that except for the mechanical difficulty of casting, the floor could be thinner. Floors have been broken during construction, but by blows which would break other floors considered amply strong. On one instance a scaffold plank fell about 16 ft., striking on end. At another time a piece of sandstone 3 ft. long, weighing about 450 lb. fell the same distance upon the floor. Each of these accidents made a break about a foot square, leaving the steel fabric but little damaged, and the repair of the floor very easy. Drilling through the floor for extending steam risers or setting anchor bolts shows the strength to be ample if not excessive.

"With the present depth of 14 in. for the cells, there is of course an economical limit to spans. Deeper cells would bring a new set of calculated values, however, so that there is no reason why long spans should not be used."

Fig. 46 is an actual view of the floor construction employed at the University of Wisconsin. Note the wood cells in the foreground ready to be removed. Note also the wood strips in the bottoms of the beams.

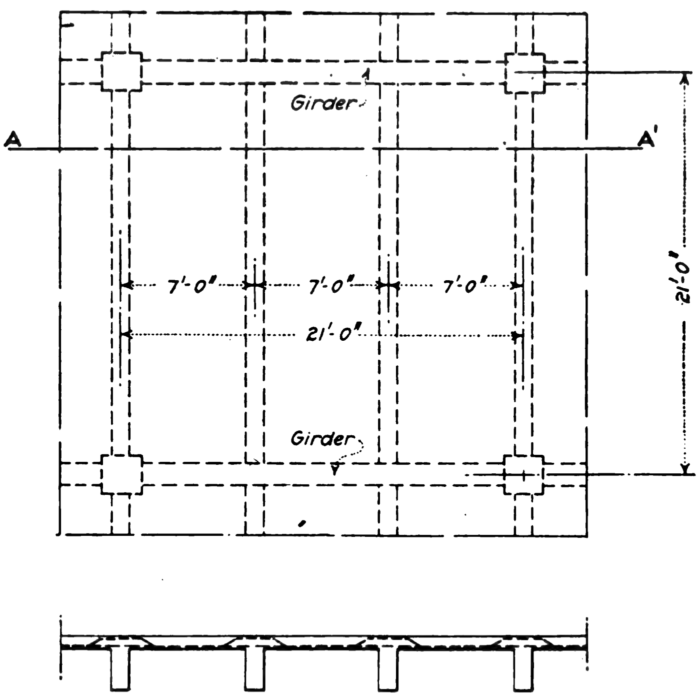
FIG. 46.—Construction view of the floor system used at the University of Wisconsin.

Floor-bay designs will now be given for both solid and hollow-tile constructions.

DESIGN OF FLOOR BAY—TWO INTERMEDIATE BEAMS

Design an interior floor bay to support a live load of 250 lb. per square foot with the columns spaced 21 ft. by 21 ft. on centers.

These floor-bay dimensions make an extra heavy construction but they will be adopted to bring out some special features in design. The working stresses recommended by the Joint Committee for a 2000-lb. concrete will be employed throughout. (See Appendix.) Plain round rods of medium steel will be assumed, and the ratio of the unit cost of steel in place to unit cost of concrete in place,[1] r, will be taken at 60.



Cross-section A-A'

FIG. 47.

Since an interior floor bay has been assumed, the bending moment $\frac{wl^2}{12}$ may be employed for the three parts of the floor bay—namely; the slab, the beam, and the girder. Fig. 47 shows the proposed arrangement of beams and girders.

The floor surface will be given what is called a granolithic

[1] The cost of concrete need not include form construction since a variation in depth affects this but slightly.

finish, consisting of a layer of 1:2 mortar, 1 in. thick, spread upon the surface of the concrete slab before it has begun to set, and troweled to a hard finish. For simplicity, the weight of finish will be assumed as included in the specified live load of 250 lb. per square foot. Of course, it should be readily understood that in practice, where a definite live load is required, the finish should be considered separately as a superimposed dead load.

The writer is aware that, under certain conditions, the mortar finish may be considered to act with the slab proper in taking the stresses under loading. Usually, however, the designer has no way of finding out whether the finish will be placed at the same time as the concrete, or run a number of hours later, and this alone should call for caution on the part of the designer in figuring the finish as a part of the slab. If the superintendent on the job is known as a careful man and if there is to be very careful supervision, the floor may be sometimes figured in this way, but never without putting an underscored note on the drawing to the effect that the finish shall be run immediately after the pouring of the concrete slab. Also the surface of the floor should be blocked off only along the center line of columns and no joint should be made between column lines, as such joints would affect the needed strength of the slab.

Where care in construction is not assured, or where any appreciable wear on the floor is expected, the finish should properly not be included in the effective slab thickness. It is also advisable not to figure this way for a winter job under any conditions.

It is possible, by taking great precautions, to bond a wearing surface to a concrete slab after the concrete in the slab has set. This requires special treatment including thorough cleaning and soaking of the old concrete, providing a bond layer of neat cement mortar, and placing the surface before this neat cement has begun to harden. Poor joints occur quite frequently, however, and it is advisable not to figure the finish as a part of the slab in a case of this kind, even though considerable care is assured at the time of construction.

If it is deemed advisable in any given design to consider the finish as an integral part of the slab, the maximum fiber stress in compression may be determined by the method outlined in Fig. 48. The distance b is made equal to the maximum allowable stress on the ordinary concrete, and the distance a which repre-

sents the maximum stress on the cement finish is then computed on this ratio. This later stress, however, should not be in excess of the maximum allowable stress for cement finish.

Slab.—The main reinforcement will be placed in the direction AA' Fig. 47, and the span of slab will be 7 ft. Slab is to be fully continuous and its total depth will be taken to the nearest $\frac{1}{2}$ in.

Table 6 of Volume I shows that a $4\frac{1}{2}$-in. ($d=3\frac{1}{2}$ in.) slab with span of 7 ft. will sustain a load (live plus dead) of $(269)(1.2) =$

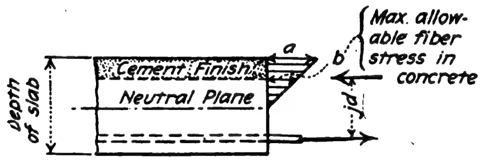


FIG. 48.

323 lb. per square foot. Corresponding weight of slab is 56 lb. per square foot, and the total load per square foot for the slab to carry is therefore $250+56=306$ lb.

For a $4\frac{1}{2}$-in. slab, Table 6 gives $a_s=0.323$ sq. in. Referring to Table 4, it is evident that $\frac{3}{8}$-in. round rods spaced 4 in. on centers will give sufficient steel area. Since the depth is taken somewhat greater than is theoretically necessary, a rod spacing of $4\frac{1}{2}$ in. will be tried. For this spacing, $p=\dfrac{0.11}{(4.5)(3.5)}=0.0070$. Using the safe load formula of Table 7, $w=296$ lb. which is less

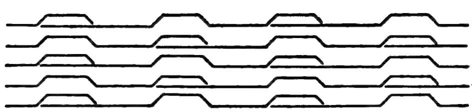


FIG. 49.

than the required load and the proposed $4\frac{1}{2}$-in. spacing cannot safely be employed unless the span is considered less than the distance between centers of supports. If desired, the span of a slab may be taken as the clear span plus the depth of slab. Four round rods $\frac{3}{8}$ in. in diameter will be placed transversely in each 7-ft. panel to prevent shrinkage and temperature cracks, and to bind the entire structure together.

Fig. 49 shows an arrangement of steel such that there is the same steel area at the top of slab over the cross-beams as at the

bottom of the slab midway between these beams. All rods are bent and are identical in shape—namely, straight at one end, bent up at the center over a beam, and bent over a beam at the other end. The required arrangement is effected by shifting alternate rods 7 ft. ahead. Some designers bend up all the rods over supports, but a better method is to continue some steel at the bottom of slab and thus make sure that no point in tension is unprovided with steel. The rods arranged as in Fig. 49 may be carried over three spans and still obtain the same amount of

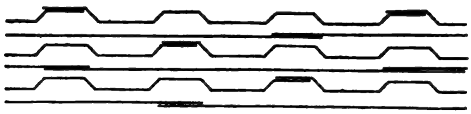


FIG. 50.

steel over supports as in the center of span, but the amount of steel at the bottom of slab near the supporting beams becomes very small.

Fig. 50 shows another arrangement for the slab steel. Both straight and bent-up rods are employed, each rod extending over three slab spans. The joints in the bent rods occur over supports and the steel is lapped a sufficient distance to provide adequate bond strength. This lapping is so arranged that two-thirds as much steel occurs over the supports as in the center of span.

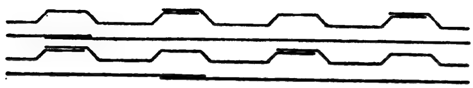


FIG. 51.

Fig. 51 shows an arrangement of steel which gives three-fourths the center-of-span area over supports. The arrangement is similar to that shown in Fig. 50 except that the rods extend over only two spans.

The arrangement shown in Fig. 50 requires the least steel and that shown in Fig. 49 requires the most. Fig. 49 shows undoubtedly the best design for spans over 6 or 7 ft.

[*Procedure using Diagrams.*—The weight of slab will be assumed at 56 lb. per square foot making a total weight (live plus dead) of 306 lb. per square foot. Diagram 5 shows the bending moment

resulting from 306 lb. for a 7-ft. span ($M=\frac{wl^2}{12}$, scale at bottom of diagram) to be 15,000 in.-lb.

Diagram 6, Part I shows that a 4½-in. ($d=3\frac{1}{2}$-in.) slab will support a load somewhat larger than that required. The area of steel is between 0.3 and 0.4 sq. in. per foot of width. Interpolation gives the value 0.31 sq. in. Diagram 3 shows this amount of area may be obtained with ⅜-in. round rods spaced 4 in. on centers.]

Cross-beams.—The cross-beams have a span of 21 ft. The beams and slab will be poured at the same time and thoroughly tied together so that a T-beam section may be considered. The distance between beams is 7 ft., and the dead and live loads of the slab per foot length of beam is equal to $7\times306=2140$ lb. Assume dead load of the stem of beam at 240 lb. per linear foot. Then the total loading per foot of length equals 2380 lb. The maximum shear

$$V=\frac{(2380)(21)}{2}=25,000 \text{ lb.}$$

and the maximum moment

$$M=\frac{(2380)(21)^2(12)}{12}=1,050,000 \text{ in.-lb.}$$

The required cross-section of web as determined by shear =

$$\frac{25,000}{105}=238 \text{ sq. in.}$$

The following formula of Art. 60, Volume I, gives the most economical depths for various assumed web widths:

$$d=\sqrt{\frac{rM}{f_sb'}}+\frac{t}{2}$$

With r in this design as 60, then

$$\begin{array}{ll} \text{for } b'=\ 9 \text{ in.} & d=23.1 \text{ in.} \\ \text{for } b'=10 \text{ in.} & d=22.0 \text{ in.} \\ \text{for } b'=11 \text{ in.} & d=21.2 \text{ in., etc.} \end{array}$$

In order to provide for most of the diagonal tension by means of bent-up rods, it is proposed to use 8 rods placed in two rows, 4 rods to a row. A value of $b'=10$ in. may be satisfactory as regards rod spacing, but $b'\times d$ as given above is not great enough to provide for shear. It will be more economical to deepen the beam of 10-in. width to a depth of 23½ in. than to adopt a width of 11 in. and a depth of about 21.2 in. Fig. 52 shows the arrange-

ment of the steel which will be tried in the bottom of the beam at the center of span. This exact arrangement will also be tried at the top of beam over supports. It is quite likely that there will be 8 rods in the girder of about 1-in or $1\frac{1}{8}$-in. diameter and, since the cross-beam rods should fit nicely with the girder rods over supporting columns, the two layers of rods will be placed 2 in. center to center as shown.

The weight of the stem is $\frac{(10)(22.5)(150)}{144} = 235$ lb. per foot, and the weight assumed is thus satisfactory.

The width of the flange of the T-beam is controlled in this design (see Art. 59, Volume I) by eight times the thickness of slab plus the width of stem, or 46 in. Then

$$\frac{M}{bd^2} = \frac{1,050,000}{(46)(23.5)^2} = 41.3$$

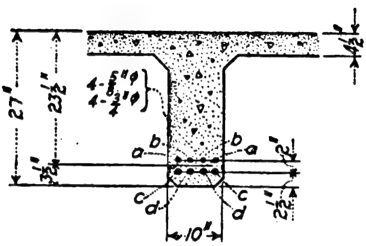
For this value of $\frac{M}{bd^2}$ and for $\frac{t}{d} = \frac{4.5}{23.5} = 0.19$, Table 9, Part I, shows p to have a value between 0.002 and 0.004, and f_c to have a satisfactory value somewhere between 297 and 484.

$$p = 0.002 + \frac{11.6}{29.1}(0.002) = 0.0028$$

$$j = 0.92$$

$$a_s = (46)(23.5)(0.0028) = 3.03 \text{ sq. in.}$$

Four $\frac{3}{4}$-in. round rods and four $\frac{5}{8}$-in. round rods will be selected (Fig. 52), having a total area of 3.00 sq. in. (See Table 13.) Eight $1\frac{1}{8}$-in. round rods would do, but the $\frac{3}{4}$-in. and $\frac{5}{8}$-in. rods are more likely to be found in stock. The four $\frac{3}{4}$-in. rods will be placed in the lower row and it will be sufficiently accurate to consider the center of gravity of the steel area as midway between the two rows. It is evident now that the width of beam for rod spacing was correctly assumed. (Note the rod spacing recommended by the Joint Committee.)



Cross-section of Crossbeam

FIG. 52.

Four rods will be bent up and lap over the top of the support. The other four will be continued straight and lap over support at

the bottom of beam. (See Fig. 53.) The bond stress along the 8 rods at the top of beam near support

$$u=\frac{25{,}000}{[(4)(2.356)+(4)(1.964)](0.85)(23.5)}=73 \text{ lb. per sq. in.}$$

which is satisfactory.

(The average value of j is taken in the above equation, see Art. 62, Volume I.) The allowable bond stress could be increased 50 per cent for the proposed arrangement of rods shown in Fig. 53. (See Arts. 38 and 63, Volume I.)

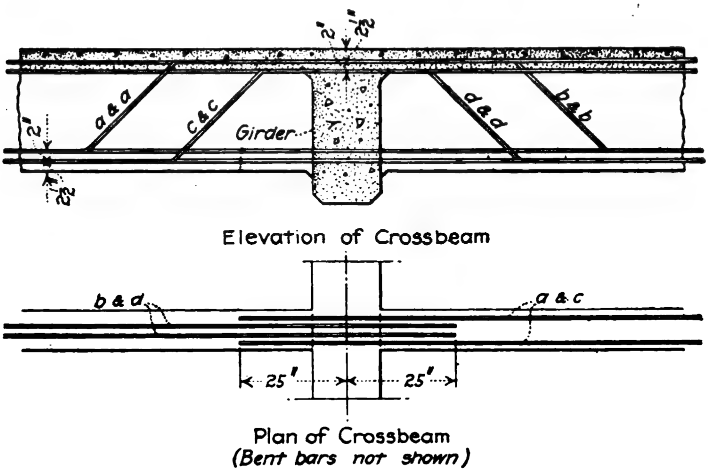


FIG. 53.

The rods at the top of beam over supports will have the same effective depth (d) as the rods at the bottom of beam at the center of span. (Figs. 53 and 54.) Then,

$$\frac{d'}{d}=\frac{3.5}{23.5}=0.149$$

$$p=\frac{3.00}{(10)(23.5)}=0.0128=p'$$

The following values may be obtained from Table 11, Part 2:

$$L=0.266$$

$$K=0.0111$$

Then,

$$f_c=\frac{1{,}050{,}000}{(10)(23.5)^2(0.266)}=715 \text{ lb. per square inch.}$$

$$f_s=\frac{1{,}050{,}000}{(10)(23.5)^2(0.0111)}=17{,}100 \text{ lb. per square inch.}$$

Allowable compression in concrete at the support may be 750 lb per square inch, and hence no haunch or extra steel is necessary. (See Art. 40, Volume I.) The tensile stress in the steel is greater than the allowable but will be considered as satisfactory here simply for the purpose of presenting several comparative designs using different numbers of rods. If a little more than the required amount of steel had been selected at the center of span—say 8 $\frac{3}{4}$-in. rounds—the stress in the steel over supports would not have figured out greater than the allowable. (See Art. 63, Volume I.)

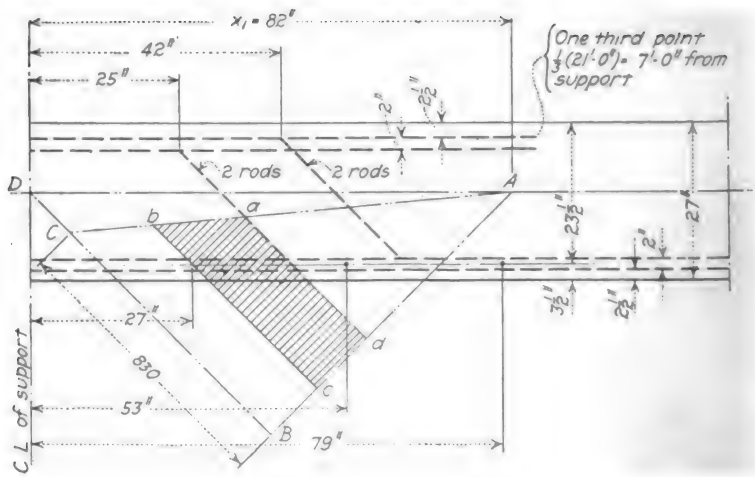


FIG. 54.

The formula given in Art. 45 of Volume I, namely,

$$x_2 = \text{or} < \frac{l}{2}\left(1 - \sqrt{\frac{8m_2}{em}}\right)$$

may be employed to find the points in the beam where the lower horizontal rods may be bent up. The first two rods may be bent at

$$\frac{21}{2}\left[1 - \sqrt{\frac{(8)(2)(0.307)}{(12)(3.00)}}\right]12 = 79 \text{ in. from center of support.}$$

The next two rods may be bent at

$$\frac{21}{2}\left[1 - \sqrt{\frac{(8)(4)}{(12)(8)}}\right]12 = 53 \text{ in. from center of support.}$$

(*Note.*—Areas are used in the first of the two preceding equations instead of the number of rods, since rods are of two different sizes.)

The rods at the top near support should be bent down as explained in Art. 63 of Volume I; the first two rods at a distance not less than $\frac{(2)(0.442)}{3.00}$ of $\frac{l}{3}=25$ in. from the center of support; and the next two at a distance ½ of $\frac{l}{3}=42$ in. from the same point. (See Fig. 54.)

The distance from the support to the point where web reinforcement is not needed,

$$x_1=\frac{21}{2}-\frac{(40)(10)(0.92)(23.5)}{2380}=6.86\text{ ft.}=82\text{ in.}$$

Also,

$$BC=\frac{2}{3}\cdot\frac{25{,}000}{(0.85)(23.5)}=830\text{ lb.}$$

Fig. 54 shows the diagonal-tension triangle. The points A and D are taken on a center line of beam, considering the depth of beam as the effective depth. This center line is approximately midway between the neutral axes for positive and negative moments.

The points to bend rods at the top of beam control the design, as shown in Fig. 54. The tensile value of one ¾-in. rod = (0.4418) (16,000) = 7070 lb. Hence stirrups will be needed to take the diagonal tension represented in the triangle ABC by the area $bcBC$, if the area $abcd$ is made equal to 7070 lb.[1] This is due to the fact that it is not reasonable to suppose that the two rods acting along da will be of use as regards diagonal tension more than one-half their value on either side. Diagonal tension near to the point A will be cared for by the additional stirrups which will be inserted to secure good T-beam action. The horizontal distance between bent rods is about the allowable—namely, about ¾d. The distance from the center of support to the point where stirrups are not necessary scales 27 in. The stirrups will be looped about the upper rods, and hence will be in an inverted position to those in a simply supported beam.

Table 15 shows that the maximum diameter of stirrups to use is (0.012)(23.5) = 0.282 in. We shall use U-shaped stirrups bent at the ends and, on account of this bending, ⅜-in. round rods will be considered secure against slipping when fully stressed. The minimum spacing of stirrups will occur at the supports,

[1] Note that the Joint Committee recommends that the stirrups be figured to take all the diagonal tension.

and this spacing (average value of $j = 0.85$, see Art. 62, Volume I)

$$s = \frac{3a_s f_s jd}{2V} = \frac{(3)(0.11)(2)(16{,}000)(0.85)(23.5)}{(2)(25{,}000)} = 4.22 \text{ in.}$$

At a distance $\frac{l}{8}$ from the center of support, $s = \frac{4}{3}$ (4.22) = 5.6 in.

At a distance $\frac{l}{4}$, $s = (2)(4.22) = 8.4$ in. A smooth curve through the points thus determined (Fig. 55) gives the spacing over the distance required. The first stirrup will be placed 2 in. from the edge of girder, assuming the girder to have a less width than

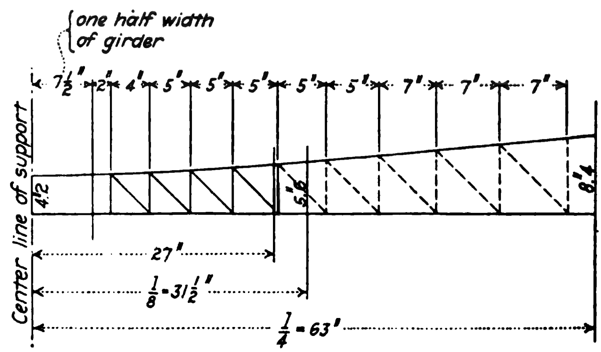


Fig. 55.

the column. In all designs the distance to the first stirrup will be made 2 in. as this is usually not greater than one-half the minimum stirrup spacing. In order to secure good T-beam action, the web and flange will be tied together with vertical stirrups placed about 18 in. on centers and looped about the lower rods for the center half of beam.

The bars bent over the support should run to the third point of the adjoining span to provide thoroughly for negative moment. The allowable stress in the compression rods at the support is $715 \times 15 = 10{,}725$ lb. per square inch, and the necessary length for bond of ¾-in. round rods is $\frac{(10{,}725)(0.75)}{(4)(80)} = 25$ in. This length is shown in Fig. 53.

[*Procedure using Diagrams.*—For the dimensions of T-beams

chosen, $\frac{M}{bd^2}=41.3$ and $\frac{t}{d}=0.19$. Diagram 8 shows $f_c=370$ lb. per square inch and $j=0.92$. Then

$$a_s=\frac{1,050,000}{(16,000)(0.92)(23.5)}=3.04 \text{ sq. in.}$$

At the supports $\frac{d'}{d}=0.149$ and $p=p'=0.0128$. For *no* compressive reinforcement, $K=\frac{M}{bd^2}=\frac{1,050,000}{(10)(23.5)^2}=190$. Diagram 1 shows the stress in the concrete to be much above 800, and it will be necessary to employ Diagram 2 and do some computing. For $p=0.0128$, $j=0.848$ and $k=0.457$. Then

$$f_s=\frac{1,050,000}{(0.0128)(0.848)(10)(23.5)^2}=17,500 \text{ lb. per square inch.}$$

$$f_c=\frac{(2)(17,500)(0.0128)}{0.457}=982 \text{ lb. per square inch.}$$

The problem is now to find what stress results in the concrete after the introduction of 1.28 per cent of compressive steel; also, what stress results in the tensile steel.

Using Diagram 10 for $\frac{d'}{d}=0.149$ and $p'=p=0.0128$, the relative reduction in the stress in the concrete is found to be 27 per cent, or the resulting stress equals $(0.73)(982)=716$ lb. per square inch. Using Diagram 11, the relative reduction in the stress in the tensile steel is about 2 per cent; that is, the maximum tension in the steel is 17,100 lb. per square inch.]

The method of designing the above beam using 6 rods will now be explained. Four $\frac{7}{8}$-in. round rods and two $\frac{5}{8}$-in. round rods will give the required area of 3.02 sq. in. The arrangement shown in Fig. 56 will be adopted.

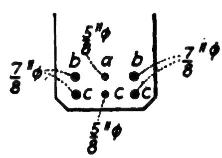


FIG. 56.

The 3 rods in the upper row will be bent up and lap over supports as shown in Fig. 57. The other 3 will lap over support at the bottom of beam. The bond stress along the 6 rods at the top of beam near support

$$u=\frac{25,000}{[(4)(2.749)+(2)(1.964)](0.85)(23.5)}=84 \text{ lb. per square inch.}$$

The allowable bond stress could be increased about 30 per cent for the arrangement of rods shown in Fig. 57.

The first $\frac{5}{8}$-in. rod may be bent at

$$\frac{21}{2}\left[1-\sqrt{\frac{(8)(0.307)}{(12)(3.02)}}\right]12=93 \text{ in. from center of support.}$$

The next two $\frac{7}{8}$-in. rods may be bent at

$$\frac{21}{2}\left[1-\sqrt{\frac{(8)(3)}{(12)(6)}}\right]12=53 \text{ in. from center of support.}$$

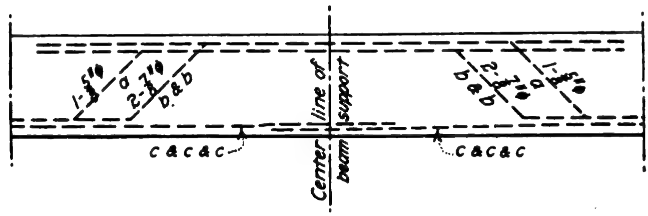


Fig. 57.

The two $\frac{7}{8}$-in. rods may be bent down at a distance not less than $\frac{(2)(0.6013)}{3.02}$ of $\frac{l}{3}=33$ in. from the center of support, and the $\frac{5}{8}$-in. rod at $\frac{1}{2}$ of $\frac{l}{3}=42$ in. from the same point.

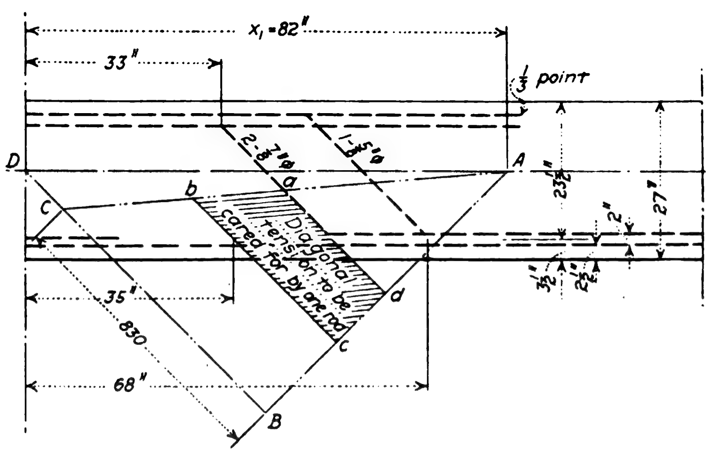


Fig. 58.

The points to bend rods at the top of beam control the location of the bends of the two $\frac{7}{8}$-in. rods. The $\frac{5}{8}$-in. rod will be so placed as to take the greatest possible amount of diagonal tension. (See Fig. 58.) The spacing of stirrups, and the distance from

the center of support to the point where stirrups are not necessary, may be found in the same manner as before.

If desired, the cross-beam may be designed using only 4 rods. Four $\frac{7}{8}$-in. square rods will give an area of 3.06 sq. in. which is only slightly more than is required. These rods will all be placed in one row as shown in Fig. 59.[1] Fig. 60 shows the complete design. Stirrups are provided to take all the diagonal tension—the strengthening action of the bent rods not being considered. The student should be able to check this design by the method outlined above. It should be noted, however, that the design is not conservative with respect to the negative-tension reinforcement, since the upper rods run only to the fourth point and the curve for negative moment has not been considered in bending down the rods. (See Page 162, Volume I.) Nevertheless a design of this kind is generally considered good. In fact, it is all that is desired under ordinary conditions in roof design because of the character and amount of the live load.

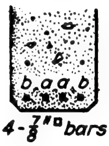


Fig. 59.

Of course, the student should realize that some latitude may generally be allowed in that part of beam design referred to

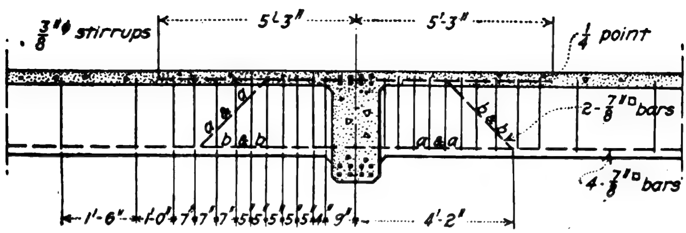


Fig. 60.

above, on account of the improbability of obtaining maximum conditions, but it is a good idea to have some conservative plan in mind, as suggested in Volume I, and to live up to that plan as nearly as circumstances will permit. At any rate, designs should never become so radical as to include rods bent up close to the support in the computations for negative reinforcement.

Fig. 61 shows a common continuous-beam design using sepa-

[1] The beam is slightly narrow according to the rod spacing recommended by the Joint Committee.

rate straight rods over the supports. All the diagonal tensile stresses are cared for by vertical stirrups.

Girder.—The girder has a span of 21 ft. with concentrated loads at the third points. The weight of the stem will be assumed at 500 lb. per linear foot. Reaction of concentrated loads $=2\times 25{,}000=50{,}000$ lb.

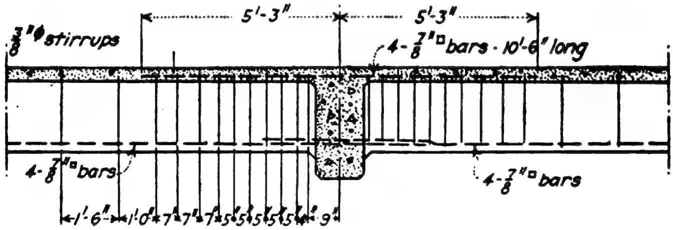


FIG. 61.

Maximum moment of concentrated loads with ends of beam simply supported would be

$$(50{,}000)(7)(12)=4{,}200{,}000 \text{ in.-lb.}$$

Taking $M=\frac{wl^2}{12}$, this moment reduces to

$$\frac{2}{3}(4{,}200{,}000)=2{,}800{,}000 \text{ in.-lb.}$$

$$\text{Moment of dead load}=\underline{220{,}500 \text{ in.-lb.}}$$

$$\text{Total moment}=3{,}020{,}500 \text{ in.-lb.}$$

This moment is fairly exact although the unsymmetrically distributed weight of a small portion of the floor slab with its live load which bears directly upon the girder has not been correctly considered.

By the method suggested in Art. 57 of Volume I, the beam may be considered as uniformly loaded with $\frac{(50{,}000)(3)}{21}+500=7640$ lb. per linear foot. The bending moment corresponding equals 3,370,000 less ten per cent. or 3,033,000 in.-lb. This moment will be employed in the computations which follow.

The total maximum shear

$$V=50{,}000+10.5(500)=55{,}300 \text{ lb.}$$

The cross-section of web as determined by shear $=\frac{55{,}300}{105}=527$ sq. in. Using the formula for economical depths, we have

b'	d	$b' \times d$
12 in.	33.1 in.	397 sq. in.
14 in.	30.8 in.	431 sq. in.
15 in.	29.8 in.	447 sq. in.
16 in.	29.0 in.	464 sq. in.

It is quite probable that a breadth of 12 in. will give proper rod spacing, but the corresponding depth of $\frac{527}{12} = 44$ in. is likely to be too great when the cost of columns and walls is taken into consideration.[1] Besides, the depth would be great in proportion to the breadth of stem and would be relatively weak at the junction of stem and flange. Illumination from windows must also be considered. The following cross-section will be taken as satisfactory:

$$b' = 15 \text{ in.} \qquad d = 32\tfrac{1}{2} \text{ in.}$$

The breadth of the flange of the T-beam is controlled in this case by eight times the thickness of slab plus the width of stem, or 51 in. Then

$$\frac{M}{bd^2} = \frac{3{,}033{,}000}{(51)(32.5)^2} = 56.3$$

For this value of $\frac{M}{bd^2}$ and for $\frac{t}{d} = \frac{4.5}{32.5} = 0.14$, Diagram 8 shows $f_c = 550$ lb. per square inch, and $j = 0.93$. Then

$$a_s = \frac{3{,}033{,}000}{(16{,}000)(0.93)(32.5)} = 6.3 \text{ sq. in.}$$

Eight 1-in. round rods (total area 6.28 sq. in.) will be chosen. The bond stress along the 8 rods at the top of beam close to the support,

$$u = \frac{55{,}300}{(8)(3.14)(0.85)(32.5)} = 80 \text{ lb. per square inch}$$

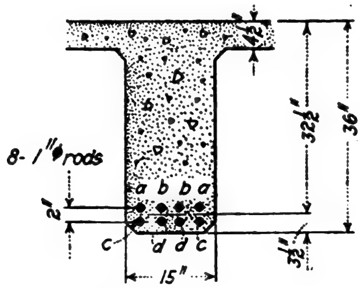
which is satisfactory.

[1] In order to include the effect of columns and walls in the formula $d = \sqrt{\frac{rM}{f_s b'}} + \frac{t}{2}$, first determine the total horizontal area covered by the stems of the T-beams under consideration and the total horizontal sectional area of the columns and walls at the level of the beams. If the cost per cubic foot of the columns and walls is greater or less than the cost per cubic foot of the T-beam stems, increase or decrease their area in proportion to the difference in cost. Then increase the cost per cubic foot of the T-beam stems in the ratio which the total corrected area bears to the area of the T-beam stems in order to obtain the unit cost c which is used to determine the ratio r.

Fig. 62 shows sketch of adopted cross-section. The weight of the stem is

$$\frac{(15)(31.5)(150)}{144} = 492 \text{ lb. per linear foot}$$

and the value assumed is on the safe side.



Cross-section of Girder

FIG. 62.

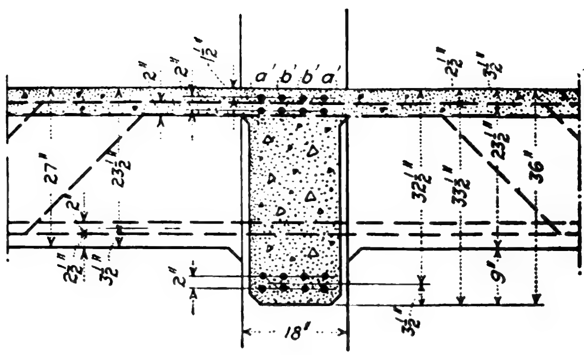


FIG. 63.

The rods at the top of girder over supporting columns will be placed as shown in Fig. 63 in order to fit in nicely with the cross-beam rods. It should be noticed that the value of d at the support is 33.5 in., or 1 in. more than at the center of span. Then

$$\frac{d'}{d} = \frac{2.5}{33.5} = 0.075$$

$$p = \frac{6.28}{(15)(33.5)} = 0.0125$$

$$p' = 0.5\,p \text{ (see Fig. 64)}$$

The following values are obtained from Table 11, Part 1:

$$L = 0.244$$
$$K = 0.0111$$

Then

$$f_c = \frac{3,033,000}{(15)(33.5)^2(0.244)} = 740 \text{ lb. per square inch.}$$

$$f_s = \frac{3,033,000}{(15)(33.5)^2(0.0111)} = 16,200 \text{ lb. per square inch.}$$

These values will be considered satisfactory.

It is proposed to have bent rods take as much of the diagonal tension as possible. The total maximum shear = 55,300 lb. The shear on the support side of the third point = 55,300 − 500 (7) = 51,800 lb. On the side of the third point toward the center of

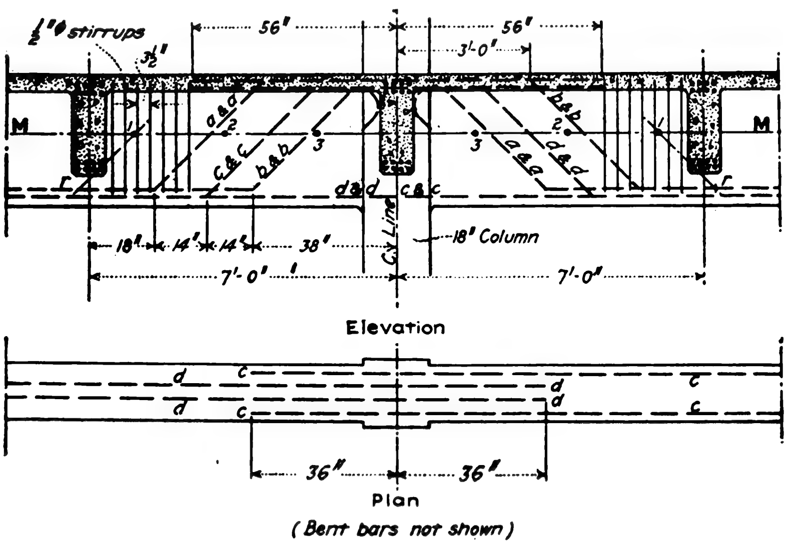


FIG. 64.

span, the shear = 51,800 − 50,000 = 1800 lb., or $v = 4$ lb. per square inch. Thus, web reinforcement is needed only from the support to the point where the beam intersects the girder.

Horizontal shear (measures diagonal tension) at the support

$$\frac{V}{jd} = \frac{55,300}{(0.85)(33.5)} = 1950 \text{ lb. per linear inch}$$

and at the third point, it is

$$\frac{51,800}{(0.93)(32.5)} = 1720 \text{ lb. per linear inch.}$$

The total diagonal tension is represented by a trapezoid, the parallel sides of which are 1950 lb. and 1720 lb., and the length 7 ft. Hence, total diagonal tension is

$$\frac{1950+1720}{2}(7.0)(12)=154,100 \text{ lb.}$$

Two-thirds of this amount, or 102,700 lb., will be taken by the web reinforcement. If six rods are to be bent, their tensile value is

$$\frac{(6)(0.785)(16,000)}{0.7}=108,000 \text{ lb.}$$

which is in excess of the stress to be provided for.

Now shear is nearly uniform between the supports and the third point, and, as far as diagonal tension is concerned, it would be sufficiently accurate to give equal spacing to the inclined rods. Since the size of columns is not given, an 18-in. diameter of column will be considered. The spacing suggested above, then, would be taken between a point 9 in. from the center of support (that is, at the edge of column) and a point where the center of the beam intersects the girder. The plan proposed is to bend 6 rods, two at a time, and the points to bend for diagonal tension should be laid off on a line approximately midway between the neutral axes for positive and negative moment—as MM, Fig. 64. These points (1, 2, and 3) may be determined by dividing the distance mentioned above into three equal parts and locating a point at the center of each part.

An investigation must now be made to determine whether or not the tensile stresses in the beam will permit the bending of the rods as above suggested. From a study of moment curves of continuous beams loaded at the third points, for different conditions, it is found sufficiently on the safe side, as regards bending up rods, to consider the point of zero moment to occur at a distance $\frac{2}{3}$ of $\frac{l}{3}$ from the third point measured toward the support. (A study of this kind will be presented in Art. 68.) Also, when considering the bending down of rods, the same distance, or $\frac{2}{3}$ of $\frac{l}{3}$ (56 in. in this case), may be safely taken as the distance out from the support to the point of zero moment. The curve of bending moments in each case is to all practical purposes a straight line. Thus, the point where the first 2 rods may be bent up, using the above data, is about $\frac{(56)(2)}{8}=$ 14 in. from the center of the

intersection of the cross-beams. Allowing say 4 in. beyond the theoretical point for bending, this distance becomes 18 in.

As regards diagonal tension, the rod to intersect the center line at point 1 should be bent at r, as shown by the dotted line. Since rods cannot be bent at r, stirrups will be employed to take the diagonal tension between the bent rods and the cross-beam. (Stirrups will also be placed at occasional intervals throughout the girder to bind together the web and flange.) Round rods of ½-in. diameter may be used for stirrups if bent at the upper end. The tensile value of each stirrup (U-shape) is $(2)(0.196)(16{,}000)=6270$ lb. The shear to be provided for in 1-in. length of beam is 1720 lb. and it will be necessary to space the stirrups $\frac{6270}{1720}=3.6$ in., say 3½ in., apart as shown in Fig. 64.

It should be noticed that in bending up the lower rods attention should be paid to the points where the upper rods may be bent down. In the design at hand, using 45 degree angle bends, the rods may be bent approximately as planned and the design will be accepted. The horizontal spacing of the bent rods should not be greater than the distance between points 1 and 2, or between points 2 and 3, unless stirrups are provided where such spacing occurs.

The rods at the top of girder should extend each side of the center of support far enough to obtain their full strength in bond, which is 50 in.

The maximum stress in the compression rods at the support is $740\times15=11{,}100$ lb. per square inch. The necessary length for bond of 1-in. rods is $\frac{(11{,}100)}{(4)(80)}=35$ in., say 36 in. This length is is shown in Fig. 64.

The top of the slab over the girder will be reinforced transversely with ⅜-in. rods spaced 12 in. center to center, in order to provide for the negative bending moment produced with the bending of the slab next to the girder.

Plates V to VIII inclusive give different complete designs for the 21-ft. by 21-ft. floor bay in question. If desired, all the cross-beam and girder reinforcement may be made into frames, except the cross-beam reinforcement running into columns in the designs of Plates V and VII. Even for these beams, however, the stirrups may be rigidly spaced if wired at the upper turns to

longitudinal rods of small diameter. After the stirrup steel is suspended in the forms, the bent rods can then be easily slipped in place and wired.

Fig. 65 and Plate VI show a girder design such that both girder and cross-beam reinforcement may be built into frames. In the arrangement shown, the girder frames (with the exception of the rods over supports) would be put in place first, then the cross-beam frames running into columns, next the negative tension rods of the girders, and finally the two intermediate cross-beam frames. The arrangement as regards strength is not as ideal as in the girder design of Plates V, VII, and VIII since the bent-up rods do not extend over supports, but in the

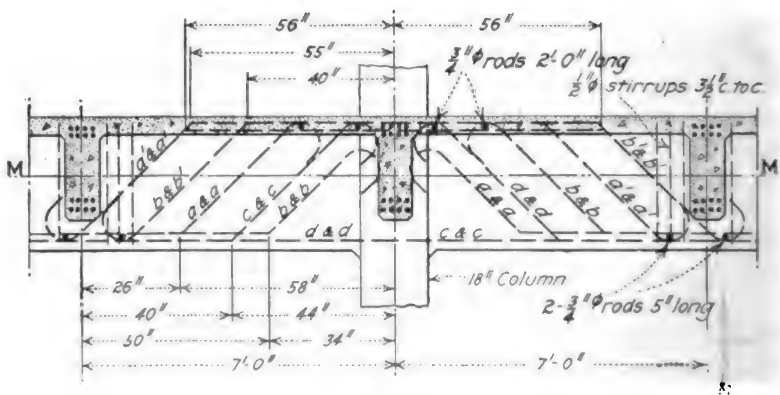


Fig. 65.

design shown there is a surplus of diagonal-tension reinforcement which offsets this weakness. To be sure the bent rods are anchored by transverse rods placed between layers of steel, but the danger lies in the liability of these bars to slip horizontally when a force inclined to the vertical is exerted upon them. It seems to the writer, however, that the additional amount of diagonal-tension reinforcement over and above that figured will make the design a safe one. The liability to slip applies more especially to the lower transverse rods, as the upper rods are long and well anchored in the slab.

If the cross-beam steel in Plate VII could have been placed over the top of the girder steel at columns, then all reinforcement shown on this plate could have been made up into frames before being placed in the forms. The reason why the cross-beam rods

were not placed above the girder rods in the design in question was because the cross-beam rods would then interfere with the main reinforcement of the slab. To prevent this, the girder rods over supports would need to be lowered and this could not be accomplished in the design in question because of the compressive stress in the concrete being already a maximum at this point. It could be accomplished, however, by either forming a flat haunch, by inserting extra compressive steel, or by deepening the girder. A similar cross-beam design to that shown on Plate VII, but where the upper reinforcement passes over the top of the girder steel at columns, will be given in Art. 38 in connection with the design of an interior roof bay.

Placing of the reinforcement in the forms in frames insures accurate location of all steel. If a loose rod method is used, great care is required in construction to make sure that the steel is not disturbed during the pouring of the concrete. Usually small diameter rods are needed to make the frames, in addition to the main reinforcement.

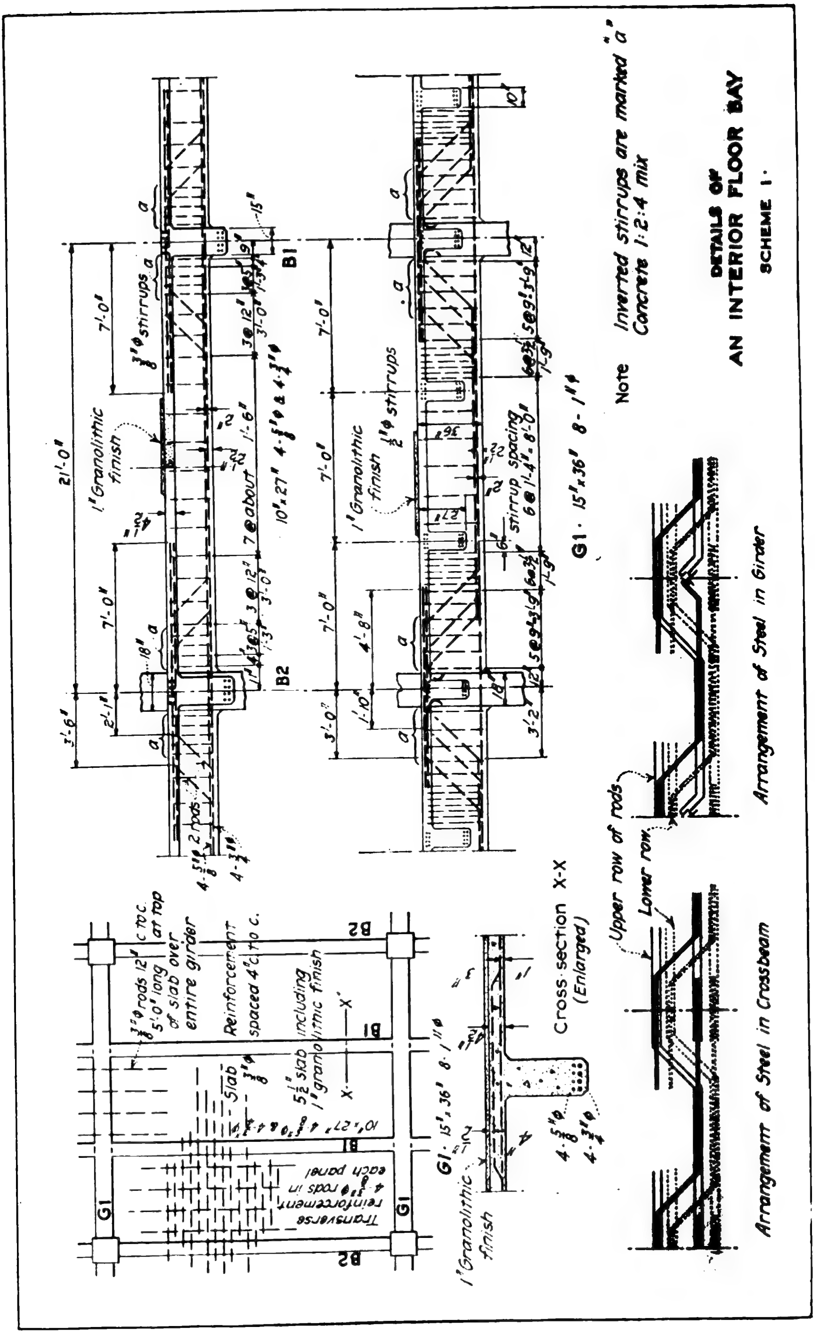
The placing of inverted stirrups near the supports has not been considered in the above discussion. These are placed after all other reinforcement is in its proper position and are slipped down over the negative-tension steel and wired to it. The continuous stirrup shown in some of the designs is convenient for this purpose. In schemes 3 and 4 the continuous stirrup is also used in place of some of the upright U-stirrups.

Spacing rods should be placed in all beams and girders between the upper and lower rows of steel. These are plain rods of the desired size and should be placed transversely not more than 5 ft. on centers. Special frame supports may be provided, if desired, or U-shaped stirrups may be employed if the length of hook is made sufficient to permit the stirrups to rest on the slab form. If special supports are used, they should be spaced about 5 ft. on centers. In the steel schedules given, special frame supports and spacing rods are omitted for simplicity.

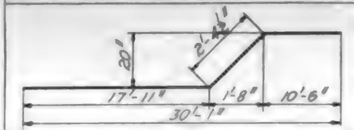
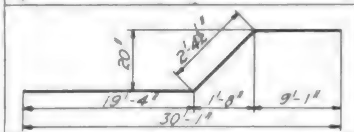
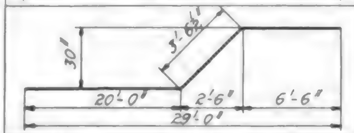
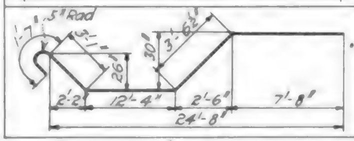
The width of stirrups is given in the bending schedules as the clear width inside of the outer strands, and the vertical height is given inside the turns. A little thought will make clear that these are the dimensions needed in bending.

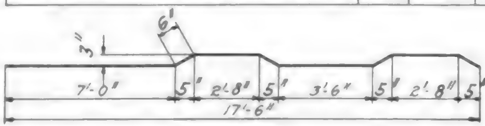
In the building design to follow, the steel arrangement on Plate V is employed. As already explained this is not entirely a frame system, but it is thought that the design is so much

PLATE V


Note Inverted stirrups are marked "a"
Concrete 1:2:4 mix
DETAILS OF
AN INTERIOR FLOOR BAY
SCHEME 1
G1 · 15" x 36" 8 - 1"φ
10" x 27" 4-5/8"φ & 4-3/4"φ
Cross-section X-X
(Enlarged)
Arrangement of Steel in Girder
Arrangement of Steel in Crossbeam
Upper row of rods
Lower row
1" Granolithic finish
Transverse reinforcement 4-3/8"φ rods in each panel
Reinforcement spaced 4" c. to c.
5½" slab including 1" granolithic finish
3/8"φ rods 12" c. to c. at top 5'-0" long of slab over entire girder
Stirrup spacing
B1
B2
G1

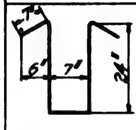
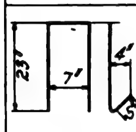
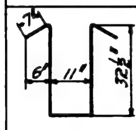
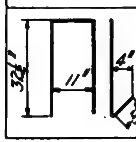
BEAM AND GIRDER RODS

Bends	Size	Length	For beam or girder Mk.	No.	Number wanted for each beam	Total number wanted
	$\frac{5}{8}''\phi$	$30'-9\frac{1}{2}''$	B1	2	4	12
			B2	1	4	
	$\frac{3}{4}''\phi$	$30'-9\frac{1}{2}''$	B1	2	4	12
			B2	1	4	
	$1''\phi$	$30'-0\frac{1}{2}''$	G1	1	4	4
	$1''\phi$	$28'-2\frac{1}{2}''$	G1	1	4	4



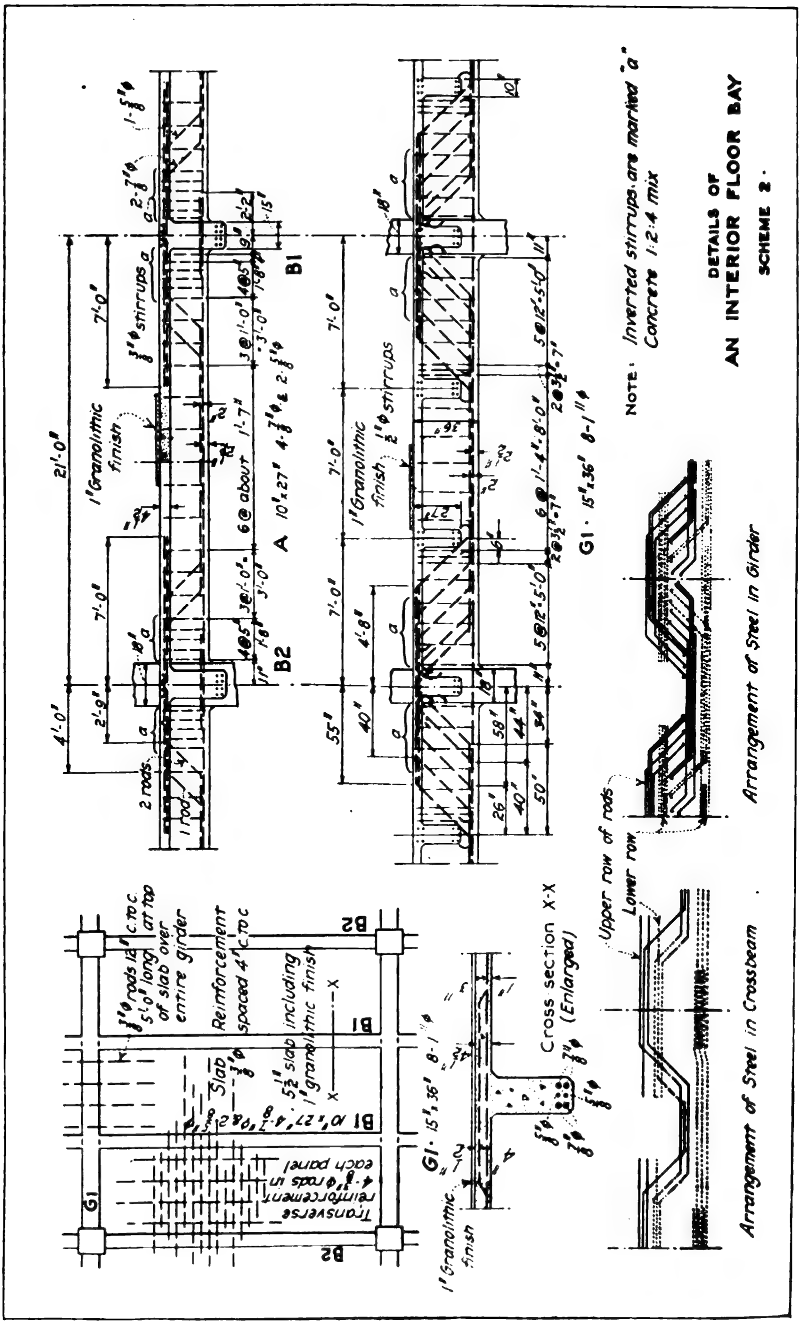
$\frac{3}{8}''\phi$ Slab Steel Total length $17'-10''$

STIRRUPS

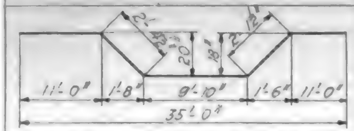
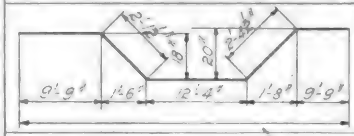
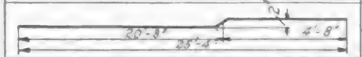
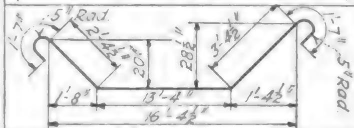
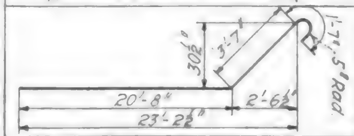
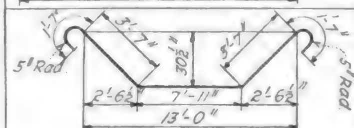
Bends	Size	Length	For beam or girder Mk.	No.	Number wanted for each beam	Total number wanted
	$\frac{3}{8}''\phi$	$5'-9''$	B1	2	12	36
			B2	1	12	
	$\frac{3}{8}''\phi$	$5'-3''$	B1	2	10	30
			B2	1	10	
	$\frac{1}{2}''\phi$	$7'-6''$	G1	1	21	21
	$\frac{1}{2}''\phi$	$7'-2''$	G1	1	8	8

STEEL BENDING SCHEDULE
FOR
AN INTERIOR FLOOR BAY
SCHEME I

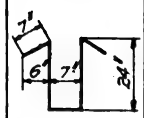
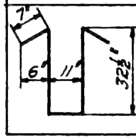
PLATE VI


DETAILS OF
AN INTERIOR FLOOR BAY
SCHEME 2
NOTE: Inverted stirrups are marked "a"
Concrete 1:2:4 mix
Arrangement of Steel in Girder
Arrangement of Steel in Crossbeam
Upper row of rods
Lower row
Cross section X-X
(Enlarged)
1" Granolithic finish
Transverse reinforcement
each panel
Reinforcement spaced 4" c. to c.
entire girder
B1
B2
G1
A
21'-0"
7'-0"
4'-0"
55"

BEAM AND GIRDER RODS

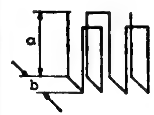
Bends	Size	Length	For beam or girder Mk	No	Number wanted for each beam	Total number wanted
	$\frac{5}{8}$"φ	36'-4"	B1	2	1	3
			B2	1	1	
	$\frac{7}{8}$"φ	36'-4"	B1	2	2	6
			B2	1	2	
	$\frac{5}{8}$"φ	25'-4"	B1	2	1	3
	$\frac{7}{8}$"φ	25'-4"	B2	1	2	6
	1"φ	22'-3"	G1	1	4	4
	1"φ	25'-10"	G1	1	4	4
	1"φ	18'-3"	G1	1	4	4
Straight	1"φ	9'-4"	G1	1	4	4

U-STIRRUPS

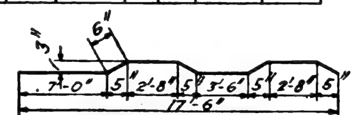
Bends	Size	Length	Beam or Girder Mk	No.	Number for each beam	Total No. wanted
	$\frac{3}{8}$"φ	5'-9"	B1	2	11	33
			B2	1	11	
	$\frac{1}{2}$"φ	7'-6"	G1	1	13	13

CONTINUOUS STIRRUPS

a	b	Spacers	Size	Length	Beam Mk	No.	No. for each beam	Total No. wanted
24	7	4-4@5	$\frac{3}{8}$"φ	29'-6"	B1	2	2	4
		4@5		24'-7"	B2	1	2	2
$32\frac{1}{2}$	11	3@12	$\frac{1}{2}$"φ	28'-4"	G1	1	2	2



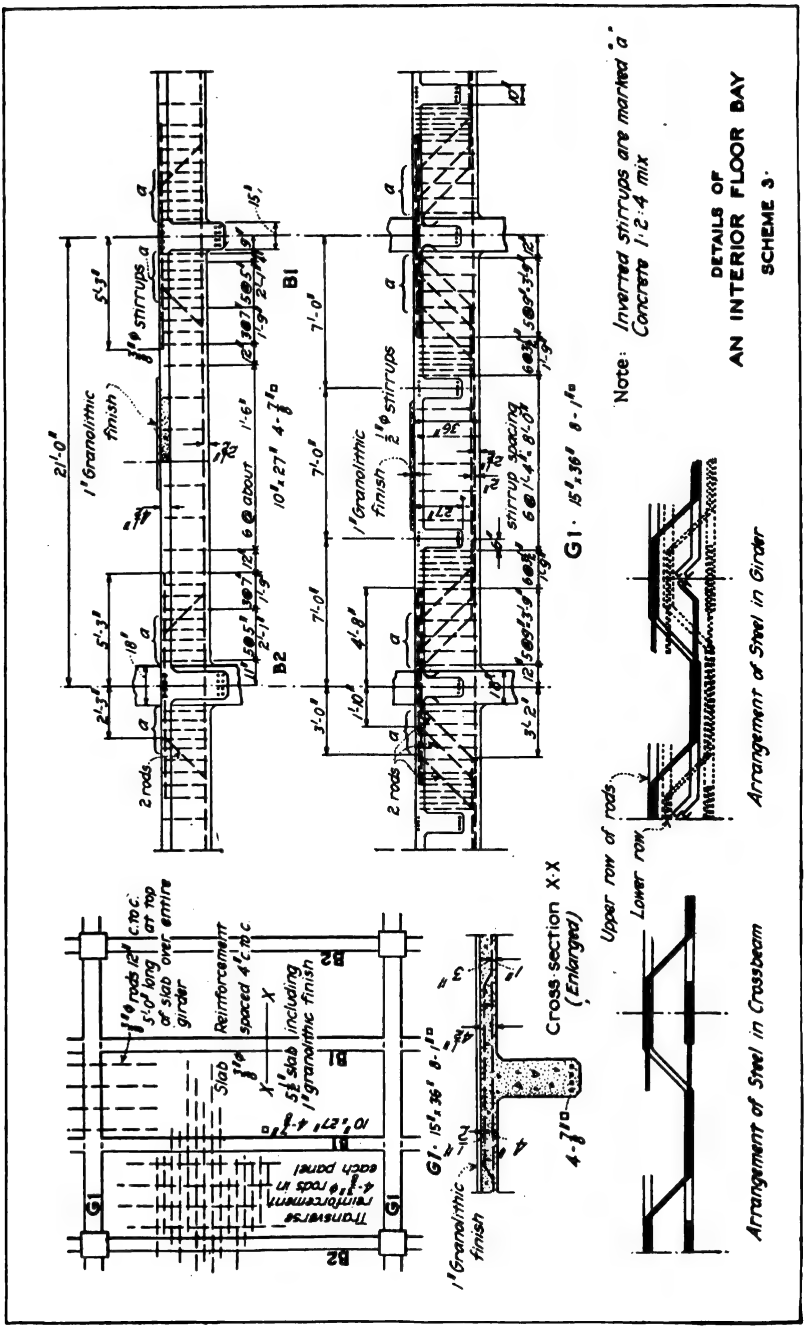
Continuous Stirrup



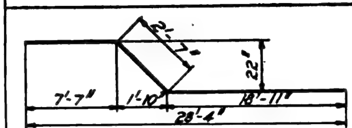
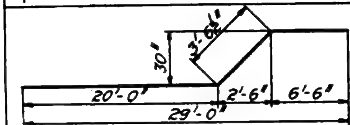
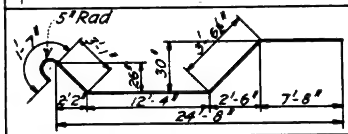
$\frac{3}{8}$"φ Slab Steel 17'-10" long

STEEL BENDING SCHEDULE
FOR
AN INTERIOR FLOOR BAY
SCHEME 2

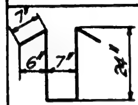
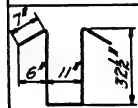
PLATE VII


DETAILS OF
AN INTERIOR FLOOR BAY
SCHEME 3
Note: Inverted stirrups are marked 'a'
Concrete 1:2:4 mix
B1
B2
G1
1" Granolithic finish
Cross-section X-X
(Enlarged)
Transverse reinforcement
Reinforcement spaced 4" c. to c.
Slab
Upper row of rods
Lower row
2 rods
stirrup spacing
Arrangement of Steel in Girder
Arrangement of Steel in Crossbeam

BEAM AND GIRDER RODS

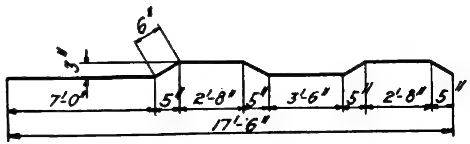
Bends	Size	Length	For beam or girder Mk.	No.	Number wanted for each beam	Total number wanted
	$\frac{7}{8}''$ □	29'-1"	B1	2	4	12
			B2	1	4	
	1" □	30'-0½"	G1	1	4	4
	1" □	28'-2½"	G1	1	4	4

U-STIRRUPS

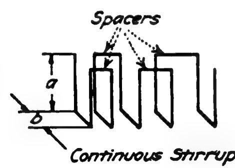
Bends	Size	Length	For beam or girder Mk.	No.	Number wanted for each beam	Total number wanted
	$\frac{3}{8}''\phi$	5'-9"	B1	2	15	45
			B2	1	15	
	$\frac{1}{2}''\phi$	7'-6"	G1	1	5	5

CONTINUOUS STIRRUPS

a	b	Spacers	Size	Length	For beam or girder		Number for each beam	Total number wanted
24	7	4-4@5	$\frac{3}{8}''\phi$	29'-6"	B1	2	2	6
		4@5		24'-7"	B2	1	2	
32½	11	9-3½-3½-3½-3½-3½-3½	$\frac{1}{2}''\phi$	53'-2"	G1	1	2	2
		3@9'		27'-7"			2	2



$\frac{3}{8}''\phi$ Slab Steel Total length 17'-10"



STEEL BENDING SCHEDULE
FOR
AN INTERIOR FLOOR BAY
SCHEME 3

PLATE VIII

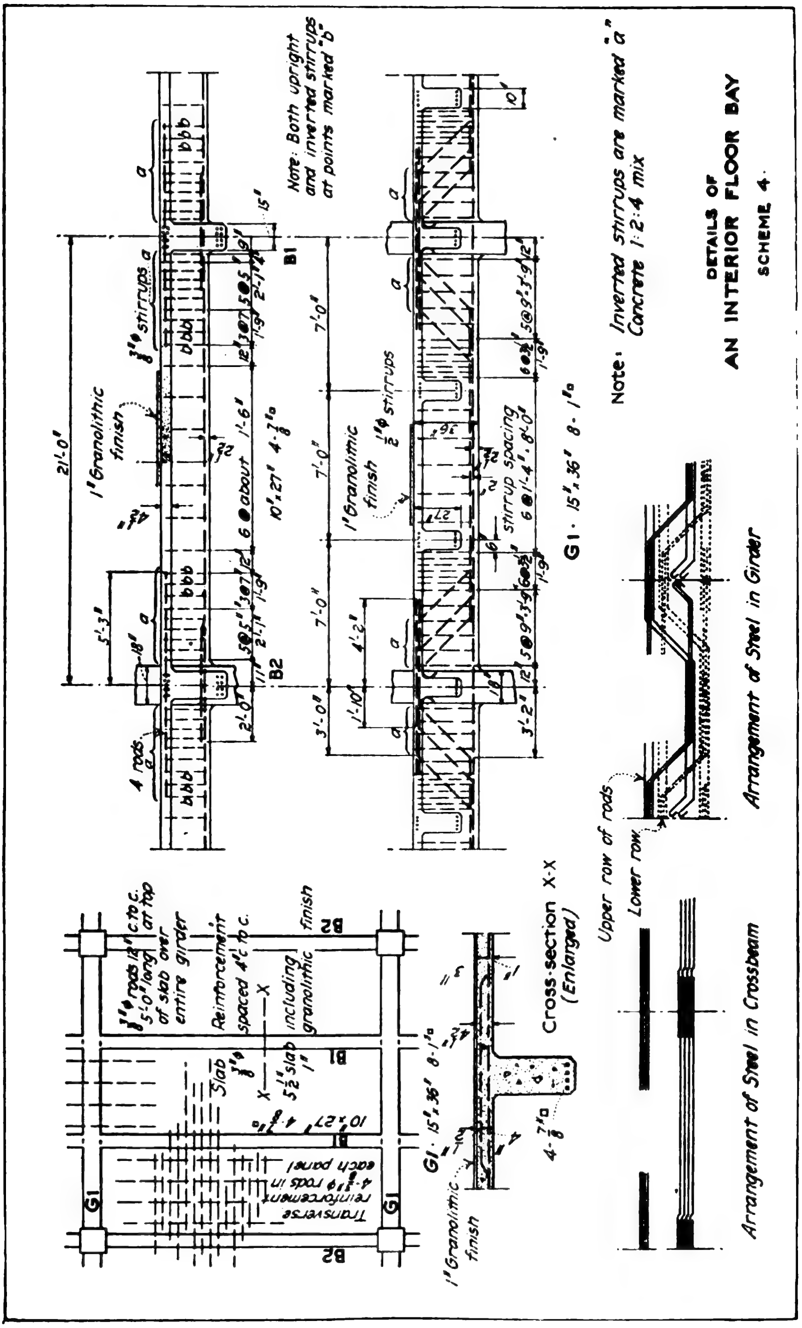
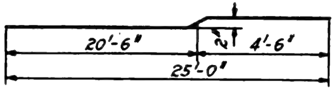
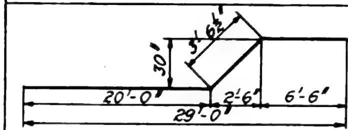
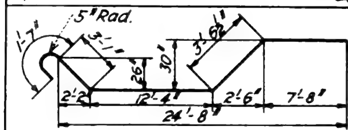
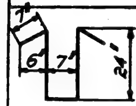
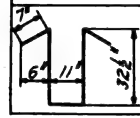

DETAILS OF
AN INTERIOR FLOOR BAY
SCHEME 4
Note: Inverted stirrups are marked "a"
Concrete 1:2:4 mix
Note: Both upright and inverted stirrups at points marked "b"
21'-0"
1" Granolithic finish
3/8"φ stirrups
1/2"φ stirrups
stirrup spacing
10"x27" 4-7/8"□
G1 · 15"x36" 8-1"□
4 rods
Cross-section X-X (Enlarged)
Transverse reinforcement 4-3/8"φ rods in each panel
3/8"φ rods 12" c. to c. 5'-0" long at top of slab over entire girder
Reinforcement spaced 4" c. to c.
Slab 3/8"φ
5½" slab including 1" granolithic finish
Upper row of rods
Lower row
Arrangement of Steel in Girder
Arrangement of Steel in Crossbeam

PLATE VIII (A)

BEAM AND GIRDER RODS

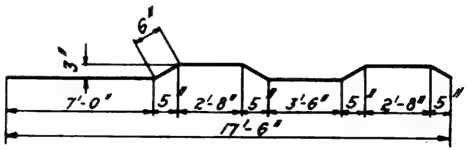
Bends	Size	Length	For beam or girder Mk.	No.	Number wanted for each beam	Total No. wanted
Straight	$\frac{7}{8}$"□	10'-6"	B1	2	4	12
			B2	1	4	
	$\frac{7}{8}$"□	25'-0"	B1	2	4	12
			B2	1	4	
	1"□	30'-0$\frac{1}{2}$"	G1	1	4	4
	1"□	28'-2$\frac{1}{2}$"	G1	1	4	4

U-STIRRUPS

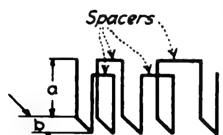
Bends	Size	Length	For beam or girder Mk.	No.	Number for each beam	Total No. wanted
	$\frac{3}{8}$"ϕ	5'-9"	B1	2	15	45
			B2	1	15	
	$\frac{1}{2}$"ϕ	7'-6"	G1	1	5	5

CONTINUOUS STIRRUPS

a	b	Spacers	Size	Length	Beam or Girder Mk.	No.	No. for each beam	Total No. wanted
24	7	4-5@5-3@7	$\frac{3}{8}$"ϕ	50'-0"	B1	2	2	6
		5@5-3@7		45'-1"	B2	1	2	
32$\frac{1}{2}$	11	9-3$\frac{1}{2}$-3$\frac{1}{2}$-3$\frac{1}{2}$-3$\frac{1}{2}$-3$\frac{1}{2}$-3$\frac{1}{2}$	$\frac{1}{2}$"ϕ	53'-2"	G1	1	2	2
		3@9		27'-7"			2	2



$\frac{3}{8}$"ϕ Slab Steel · Total length 17'-10"



Continuous Stirrup

STEEL BENDING SCHEDULE
FOR
AN INTERIOR FLOOR BAY
SCHEME 4

superior to that shown on Plates VI, VII, and VIII that the inconvenience of using 8 rods in the cross-beam and placing some of the rods loose will be more than offset by the ideal conditions as regards strength obtained in both cross-beam and girder. The cross-beam reinforcement could be made up into frames in some such manner as illustrated in Fig. 66. The girder steel could be arranged in a somewhat similar fashion.

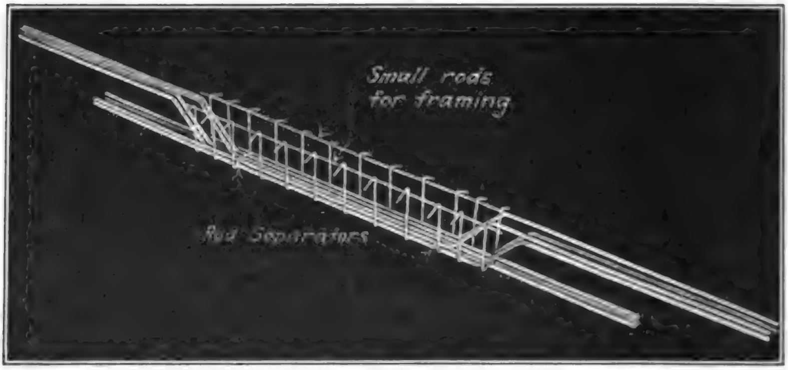


FIG. 66.

The writer realizes that the frames employed would be long and heavy and that they would need to be built close to the forms in which they would be placed, but it would seem that any inconvenience in the placing of the frames would also be offset by the excellence of the design.

PROBLEM

5. Make a complete design of an interior floor bay with a 1-in. granolithic finish to support a live load of 200 lb. per square foot, columns to be spaced 18 ft. by 20 ft. on centers. The girders are to span 18 ft. and the distance between centers of cross-beams is to be made 6 ft. Use working stresses recommended by the Joint Committee for a 2000-lb. concrete and arrange details as shown in illustrative designs. Assume plain round rods of medium steel. Take ratio of unit cost of steel in place to cost of concrete in place, $r=60$. The 1-in. finish is not to be included as a part of the effective slab thickness and its weight is to be added to the dead load of slab.

DESIGN OF FLOOR BAY—ONE-WAY HOLLOW TILE

Design an interior panel of a one-way hollow tile floor to carry a live load of 100 lb. per square foot with the rows of girders spaced

21 ft. on centers. As in the preceding design, the recommendations of the Joint Committee will be followed for a 2000-lb. concrete. The ratio of the unit cost of steel in place to unit cost of concrete in place (r) will be taken at 70, with 20 cents as the cost of concrete per cubic foot.

The finished flooring will consist of $\frac{7}{8}$-in. maple boards nailed to 2 ×3-in. sleepers. The sleepers will be placed on the concrete slab and cinder concrete in proportions 1:3:6 filled in between them. The following weights will be taken in pounds per square foot of floor area: wooden floor, 5; sleepers or nailing strips, 2; concrete filling, 15; plaster, 5.

Ribs 4 in. wide will be assumed making a 16-in. width of flange for the small T-beams. The concrete topping will be made 2 in. If a 9-in. tile is assumed, the total load per linear foot of beam will be 274 lb., made up of the following items:

$$
\begin{aligned}
\text{live load} &= 100 \times \frac{16}{12} = 133 \text{ lb.} \\
\text{wood floor} &= 5 \times \frac{4}{3} = 7 \text{ lb.} \\
\text{sleepers} &= 2 \times \frac{4}{3} = 3 \text{ lb.} \\
\text{concrete filling} &= 15 \times \frac{4}{3} = 20 \text{ lb.} \\
\text{concrete topping} &= 25 \times \frac{4}{3} = 33 \text{ lb.} \\
\text{tile} &= 33 \text{ lb.} \\
\text{stem} &= \frac{(4)(9)}{144}(150) = 38 \text{ lb.} \\
\text{plaster} &= 5 \times \frac{4}{3} = 7 \text{ lb.} \\
\text{Total} &= 274 \text{ lb.}
\end{aligned}
$$

The bending moment,

$$M = \frac{wl^2}{12} = \frac{(274)(21)(21)(12)}{12} = 121{,}000 \text{ in.-lb.}$$

The economical depth of floor now needs to be determined. Using the same notation as in Art. 60 of Volume I and, in addition, using the term c_t to represent the variation in cost of tile in place per 1-in. change in depth, we have

$$C = cb'd' + \frac{crM}{f_s\left(d' + \frac{t}{2}\right)} + d'c_t$$

as the total cost of the small T-beams per unit length. The following expression has been deduced from the preceding equation by the aid of the Calculus and will give the value of d for minimum cost when the value of b' is fixed:

$$d=\sqrt{\frac{c_c rM}{f_s(b'c_c+144c_t)}}+\frac{t}{2}$$

The term c_c in this formula means the cost of concrete per cubic foot. A value of $1\frac{1}{4}$ cents will be given to c_t. Then

$$d=\sqrt{\frac{(20)(70)(121,000)}{(16,000)(260)}}+\frac{2}{2}=7.4 \text{ in.}$$

The effective depth d must be taken so as to provide for a commercial depth of tile. A $7\frac{1}{2}$-in. effective depth will be tried with a $1\frac{1}{2}$-in. fireproof covering below the center of steel. This assumption would permit of a 7-in. tile.

Proceeding with the design, however, we find that

$$\frac{M}{bd^2}=\frac{121,000}{(16)(7.5)^2}=135$$

and

$$\frac{t}{d}=\frac{2}{7.5}=0.27$$

Diagram 8 shows the stress in the concrete to be above 650 lb. per square inch, which cannot be allowed. By trial it is found that a $9\frac{1}{2}$-in. effective depth is needed to bring the concrete stress to an allowable value. For this depth,

$$\frac{M}{bd^2}=\frac{121,000}{(16)(9.5)^2}=83.8$$

$$\frac{t}{d}=\frac{2}{9.5}=0.21$$

From Diagram 8, $j=0.91$. Then

$$a_s=\frac{121,000}{(16,000)(0.91)(9.5)}=0.88 \text{ sq. in.}$$

Two $\frac{3}{4}$-in. round rods will be employed in each rib—one will lie straight and the other will be bent up at both ends and extend along the top of beam to the quarter point of the adjoining span. This arrangement will give the same steel over supports as in the center of span.

The total shear close to support

$$V=\frac{(274)(21)}{2}=2880 \text{ lb.}$$

The bond along the two rods at the top over supports

$$u=\frac{2880}{(2)(2.36)(0.85)(9.5)}=76 \text{ lb. per square inch.}$$

The distance from the support to where stirrups are unnecessary

$$x_1=\frac{21}{2}-\frac{(40)(4)(0.91)(9.5)}{274}=5.4\text{ ft.}=65\text{ in.}$$

All the diagonal tension not taken by the concrete will be provided for by vertical stirrups; in other words, the strengthening action of the bent rod will not be considered. Round stirrups $\frac{1}{4}$-in. diameter will be employed, bent at the ends. Stirrup spacing near the support

$$s=\frac{3}{2}\cdot\frac{(0.049)(2)(16{,}000)(0.85)(9.5)}{2880}=6\tfrac{1}{2}\text{ in.}$$

The spacing adpted is shown in Plate IX.

The moment, shear, and bond considerations given above do not take into account the strengthening action of the flange of the T-shaped girders, and allowance may be made for this when thought necessary. It is quite evident that the concrete is not over-stressed in compression over supports, but the stress at the edge of girder flange should be investigated. For the width of girder flange shown in Plate IX, the bending moment may be taken at $\frac{6}{7}$ (121,000) = 104,000 in.-lb. This value is obtained by considering the point of zero moment at the third point.

$$\frac{d'}{d}=\frac{1.5}{9.5}=0.158$$

$$p'=p=\frac{0.88}{(4)(9.5)}=0.023$$

Table 11, Part 2, shows $L=0.374$ and $K=0.0195$. Then

$$f_c=\frac{104{,}000}{(4)(9.5)^2(0.374)}=770\text{ lb. per square inch.}$$

$$f_s=\frac{104{,}000}{(4)(9.5)^2(0.0195)}=14{,}800\text{ lb. per square inch.}$$

The load on the floor is $(274)\left(\frac{12}{16}\right)=205$ lb. per square foot, and the girder therefore carries a load of (205)(21) = 4300 lb. per linear foot, to which should be added the weight of the girder itself. This weight will be assumed at 360 lb. making a total load, which may be considered uniform, of 4660 lb. per linear foot. The bending moment

$$M=\frac{(4660)(21)(21)(12)}{12}=2{,}055{,}000\text{ in.-lb.}$$

Assume the total depth of girder to be limited to 36 in., effective

PLATE IX

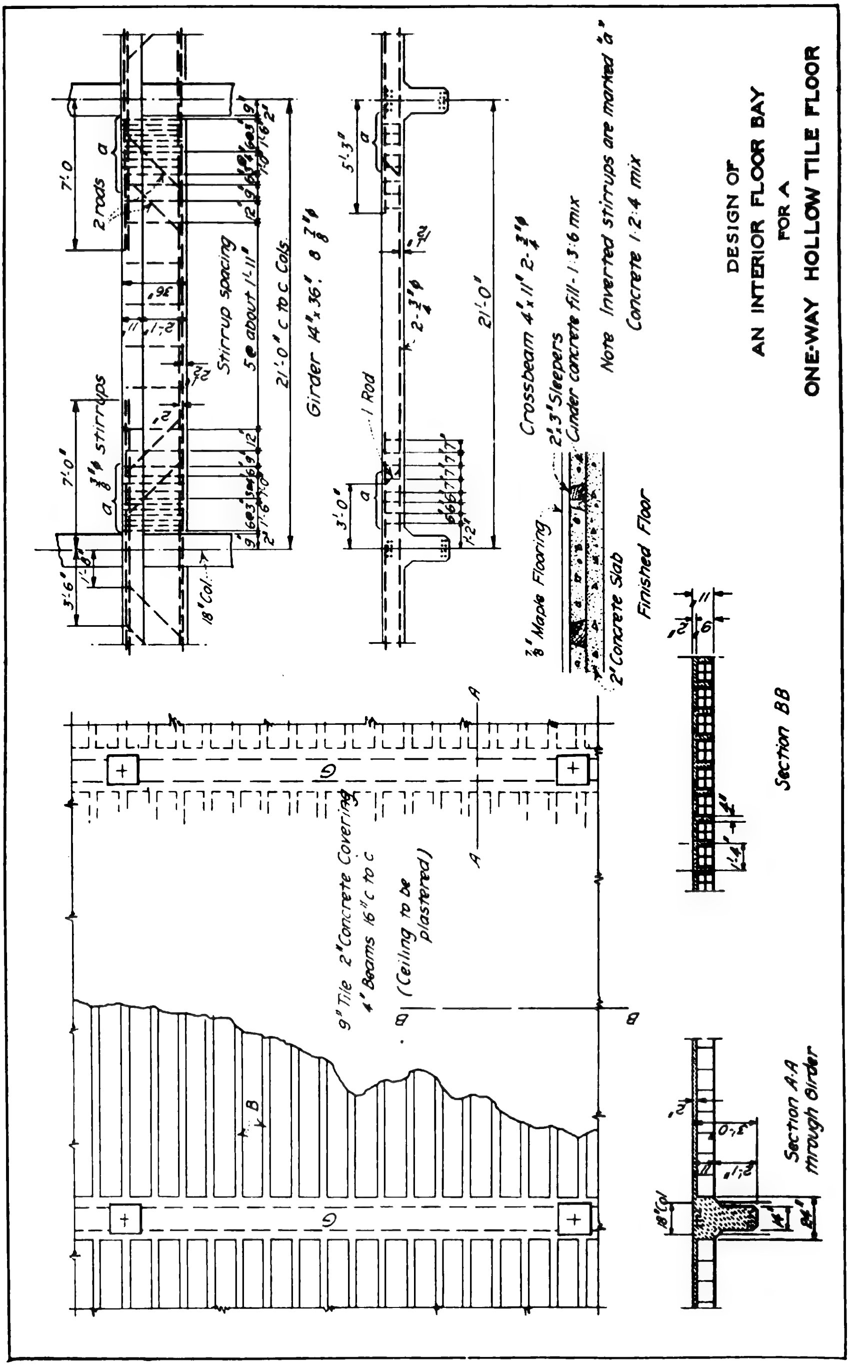
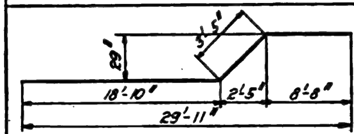
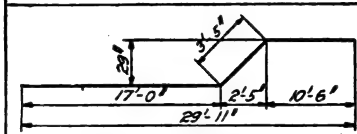
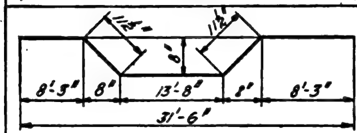
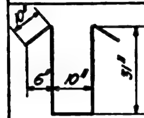
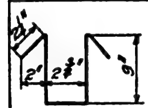

DESIGN OF
AN INTERIOR FLOOR BAY
FOR A
ONE-WAY HOLLOW TILE FLOOR
Note Inverted stirrups are marked "a"
Concrete 1:2:4 mix
Crossbeam 4"x11" 2-3/4"φ
2"x3" Sleepers
Cinder concrete fill-1:3:6 mix
7/8" Maple Flooring
2" Concrete Slab
Finished Floor
Girder 14"x36" 8 7/8"φ
Stirrup spacing
5@ about 1'-11"
21'-0" c to c Cols.
3/8"φ stirrups
2 rods
1 Rod
18" Col.
9" Tile 2" Concrete Covering
4" Beams 16" c to c
(Ceiling to be plastered)
Section BB
Section A-A through Girder

PLATE IX (A)

BEAM AND GIRDER RODS

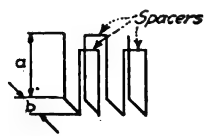
Bends	Size	Length	Beam or Girder	No. wanted for each beam	Total No. wanted
	$\frac{7}{8}''\phi$	30'-11"	G	4	4
	$\frac{7}{8}''\phi$	30'-11"	G	4	4
	$\frac{3}{4}''\phi$	32'-1"	B	1	
Straight	$\frac{3}{4}''\phi$	25'-0"	B	1	

U-STIRRUPS

Bends	Size	Length	Beam or Girder	No. wanted for each beam
	$\frac{3}{8}''\phi$	7'-8"	G	8
	$\frac{1}{4}''\phi$	$2'\text{-}1\frac{1}{4}''$	B	6

CONTINUOUS STIRRUPS

a	b		Size	Length	Beam or Girder	No. for each beam
31"	10	6@3"-3@4"-6"	$\frac{3}{8}''\phi$	69'-0"	G	2
9	$2\frac{3}{4}$	6-6-6-7	$\frac{1}{4}''\phi$	$10'\text{-}8\frac{3}{4}''$	B	2



Continuous Stirrup

STEEL BENDING SCHEDULE
FOR
AN INTERIOR FLOOR BAY
ONE-WAY HOLLOW TILE FLOOR

depth to $32\frac{1}{2}$ in. Then $\frac{t}{d}=\frac{11}{32.5}=0.34$. Diagram 8 shows that, for $\frac{t}{d}=0.34$ and $f_c=650$, $\frac{M}{bd^2}=107$, or

$$b=\frac{2,055,000}{(32.5)^2(107)}=18.2 \text{ in.}$$

An arbitrary value of 24 in. will be adopted for b, which makes $\frac{M}{bd^2}=81$. Diagram 8 shows the neutral axis to lie in the flange. Table 3 or Diagram 1 shows $p=0.0057$, or

$$a_s=(0.0057)(24)(32.5)=4.4 \text{ sq. in.}$$

Eight $\frac{7}{8}$-in. round rods will be selected, with a total area of 4.81 sq. in. The width of stem will be made 14 in. to provide properly for shear. The remaining computations for stresses in the steel and concrete at the support, and for shear and bond, are similar in every way to those given under cross-beam design in two-intermediate beam construction. In Plate IX an 18-in. column is assumed for convenience.

As in solid concrete floors, the proper depth for girders in hollow-tile construction depends upon the use to which the building is to be put, and the cost of all columns and walls in the building per unit increase in height. The cost of form work should also receive attention. Of course, as regards the materials in the girder itself, cost decreases with depth. The problem resolves itself into finding the limiting depth of the girder in view of the many conditions which must be considered. Very often a perfectly flat ceiling is desired, and very wide girders must then result.

The following table has been prepared to simplify the computations in the design of the small T-beams of a hollow-tile floor. The table shows at a glance, for any given thickness of concrete topping, the minimum depth of tile needed for any given bending moment, and the corresponding steel area required. Greater depths of tile may sometimes prove more economical.

It is quite evident from a study of the table that different thicknesses of concrete topping should be considered in order to arrive at the most economical design. When the rib width does not change in the economical considerations, the economy of the various designs may be based on the cost per foot length of floor having a width, the distance center to center of ribs; but when the steel area is such that the width of rib varies, the economy of the designs should be based on the cost per square foot of

floor. For practice, the student should work through similar computations to the foregoing using 3-in. and 4-in. toppings.

TABLE FOR HOLLOW TILE FLOORS

Based on $f_c = 650$; $f_s = 16{,}000$; $n = 15$
(1¼ in. allowed from center of steel to bottom of slab)

Bending moments and steel areas for various depths of tile and concrete (Moments in thousands of inch pounds; steel areas in square inches)

4-in. rib
For any other width of rib multiply all values in table by distance center to center of ribs and divide by 16

Depth of tile (inches)	Thickness of concrete topping (inches) 1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
4	*27/0.49	*35/0.55	*43/0.62	*52/0.68				
5	41/0.58	52/0.68	*69/0.74	*72/0.80	*84/0.86			
6	56/0.65	71/0.78	84/0.86	*96/0.92	*110/0.98	*124/1.04		
7	71/0.70	90/0.84	107/0.96	124/1.04	*139/1.10	*155/1.17	*172/1.23	
8	86/0.73	110/0.90	131/1.03	152/1.14	171/1.23	*189/1.29	*208/1.35	*227/1.41
9	104/0.76	130/0.94	156/1.09	181/1.21	204/1.32	226/1.41	248/1.48	*268/1.53
10	117/0.78	150/0.98	181/1.14	210/1.28	238/1.40	243/1.51	290/1.59	314/1.67
12	147/0.81	190/1.03	231/1.22	269/1.38	309/1.52	340/1.65	374/1.77	407/1.87

* Neutral axis in flange.

PROBLEM

6. Design an interior panel of a one-way hollow-tile floor to carry a live load of 120 lb. per square foot with the girders spaced 19 ft. on centers. Use same unit stresses and same floor construction as in the design given in the text. Make the concrete topping 2½ in. thick and use 4-in. ribs for the small T-beams. Employ an economical stem for the T-girders and choose an arbitrary value of 36 in. for the breadth of flange.

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18. Cantilever Flat-slab Construction.—The cantilever flat-slab type of floor construction was originally introduced by Mr. C. A. P. Turner of Minneapolis. The construction consists of columns and a floor slab, without beams or girders. The loads are transmitted from the floor slab directly to the columns, the head of the column being curved out at the top to increase the circumference at the line of maximum stress in the slab. The floor is usually of uniform thickness throughout.

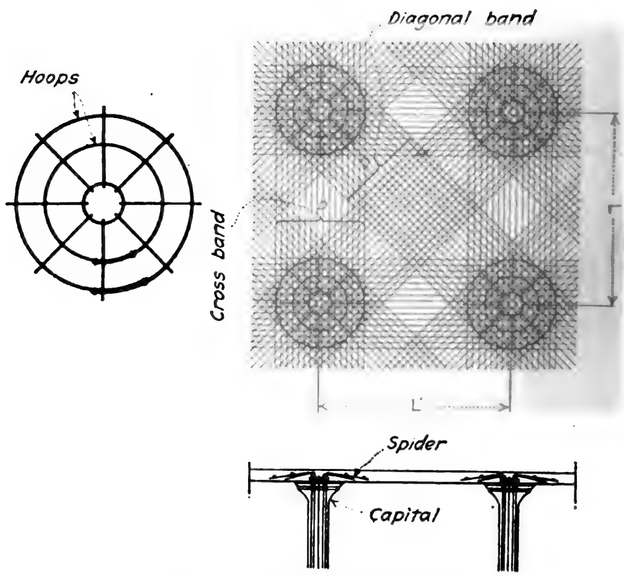


FIG. 67.—Mushroom system.

The reinforcement of the slab consists of bars running in four directions radially from the column as shown in Figs. 67 and 68. The vertical reinforcing steel in the column is sometimes bent and carried into the slab to form what is known as a spider, which aids in preventing shear and adds to the rigidity of the connection.

This is the arrangement found in the original *mushroom* type, as shown in Fig. 67.

Certain features of flat-slab reinforcement are covered by Letters Patent Nos. 985,119 and 1,003,384 of C. A. P. Turner. The following extracts cover the subject matter of these patents:

Turner patent 985,119, granted Feb. 21, 1911: "A flat-slab and column construction of concrete, including a cantilever head formed of elbow rods having portions thereof embedded in the slab, and other

Courtesy of Mr. C. A. P. Turner.

Fig. 68.—Cantilever flat-slab construction. John Deere Co.'s warehouse, Omaha, Nebraska.

portions thereof extending into the columns a distance less than the height of the columns."

Turner patent 1,003,384, issued Sept. 12, 1911: "An arrangement of reinforcement for a column-supported flat-plate floor of concrete, comprising a plurality of circumferential cantilever members, respectively situated in the upper part of the slab at the columns and projecting therefrom, and reinforcing means extending from member to member in multiple directions through the space between said members, and filling, or substantially filling, such space.

"An arrangement of reinforcement for a column-supported flat-plate floor of concrete, comprising a plurality of circumferential cantilever frames, each composed of crossed rods situated in the upper part of the slab at the columns, respectively, and extending across and outward

therefrom, and belts of rods extending from frame to frame in multiple directions, and filling or substantially filling the space between them."

The advantages claimed for the flat-slab floor, as stated by Mr. Turner, are as follows:

"By my construction, wherein a capital or enlargement is formed on the column, or in one piece therewith, and by the employment of my reinforcement, I am able to dispense with the use of beams on the under side of the floor slab. This is an immense advantage in every way. It is economical in the use of concrete; it is also economical in that it renders unnecessary the expensive forms for making the beams, and it means greater rapidity of work. As far as the finished structure is concerned, the absence of beams on the under side of the floor slab enables partitions to be placed anywhere that it may be found desirable to place them. It results in better illumination from the windows, and there are no dirt-collecting corners, which exist where beams or girders are employed.

"Another very important advantage resulting from the provision of a ceiling that is smooth, or free from beams or projections, is in the matter of fire protection. In fighting a fire with a stream of water from a hose, the obstruction offered by ribs or beams is obviously serious, since a rib may stop short a stream of water, whereas a flat, smooth surface against which the stream is directed at an angle, will deflect and spread the water, causing it to descend to the floor over a wide area and to the best possible advantage. Where sprinkler heads are used in a ceiling, the cost of equipment by such a system of fire protection is substantially reduced, because fewer sprinkler heads are required with a flat or smooth ceiling than one where there are beams or ribs on the under side of the ceiling. In warehouses, or similar buildings, my invention is of special value because in order to afford aisles or passageways, the load is naturally concentrated around the columns, and it is at these points, where the load therefore is greatest, that the greatest strength of the structure exists, by reason of the enlarged capitals of the columns, and their integral construction or formation with the slabs, and the heavy reinforcements of the structure immediately at and adjacent to the column. The provision of the capitals on the columns by gradually increasing the diameter of the columns at the top, and making them and the slab an integral mass, takes care of the compression of the concrete which is the greatest over the columns."

The structural action in the flat-slab type of floor is not fully understood and the several attempts which have been made at analysis have not met with any general acceptance. Recently, however, tests have been made of the actual structure in completed buildings and in this way considerable information has

been gained by which at least some of these methods of analysis may be controlled so as to give safe results. (See Art. 28.)

The student will get a general idea of the direction of the stresses in a flat-slab floor by referring to Fig. 69. This contour diagram appears in a pamphlet issued by the Corrugated Bar Co. of St. Louis and is the result of measurements on the deflections of a rubber plate supported by small columns and under load. The diagram shows the plate bending downward around the columns, forming contour lines approximately circular in form. The suspended-span portion of the plate is seen to have

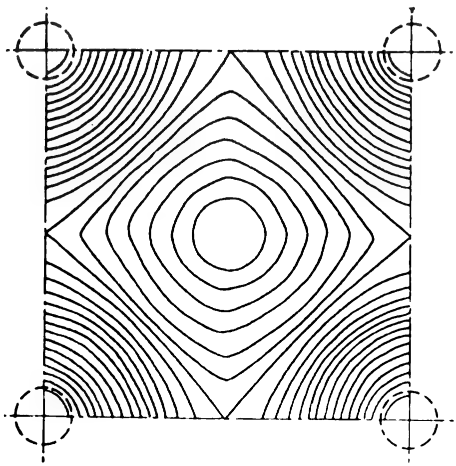


FIG. 69.

an opposite deformation, being concave downward in all directions about a center formed by the intersection of the diagonals of the panel. Fig. 70 shows that with any radial deformation, a deformation also occurs in a circumferential direction. For example, if the top fibers around the column head are elongated an amount ΔR, then the enlargement of the plate on the top causes a deformation in a circumferential direction of $2\pi \Delta R$. Thus it is evident that there are positive deformations in a flat slab under bending which have certain fixed geometrical ratios to each other.

It is contended that reinforcing rods in four directions, being tangent to the circular sections at eight points, are in better position to resist circular as well as radial stresses over supports than rods arranged in two directions. Tests, however, do not

seem to bear out this contention. Some designers advocate placing a flat spiral form of reinforcement both over the columns and in the center of panel, and combining this with the four-way reinforcement. Of course from a theoretical standpoint, sufficient reinforcement could be placed in a hooped or spiral shape so that the cross and diagonal bars could be dispensed with. This is not possible, however, in actual practice owing to the fact that concrete shrinks when setting and would pull away from the hoops; the hoops would therefore exert no pressure against the concrete until considerable, and perhaps dangerous, deformation had taken place.

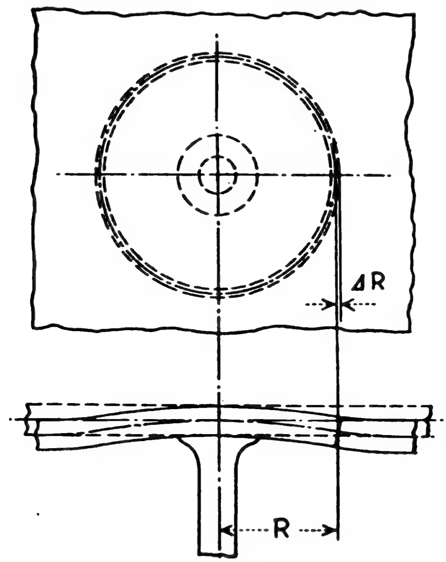


FIG. 70.

A number of methods of analysis have been proposed for the cantilever type of flat-slab construction. Perhaps the three methods which have been most generally advocated are as follows:

1. The division of the slab into beams, or beam strips, which we shall call the beam method of analysis.

2. To consider the slab around the head of the column, out to the point of inflection, as a flat circular plate supporting a uniform load over its surface and also supporting a vertical load along its edge equal to the remaining part of the load tributary to the col-

umn. In other words, the whole floor is considered as a series of flat circular slabs concentric with the columns and firmly clamped to them, and the rest of the floor is considered as supported along the edges of these circular slabs. The slab between the circular plates is considered as a square plate supported on all edges.

3. To consider the slab as a homogeneous plate fixed on a system of equidistant points.

The division of the slab into beams, or beam strips, is perhaps the most logical method of analyzing this type of floor. It must be recognized, however, that at least the steel stresses indicated by this analysis are reduced by slab action and by arch action to a very considerable degree, the amount of reduction probably depending upon the depth of the slab and upon the details of the head and slab reinforcement. Unfortunately, at the present time, it is not possible to state coefficients for this reduction. It may safely be concluded, however, that the stresses as found by this analysis will at least represent maximum values.

The second method of analysis above mentioned is based upon Prof. H. T. Eddy's analysis of stresses[1] in homogeneous circular plates. In deriving the formulas of this method, the effect of lateral stresses has been taken into account, this being expressed by Poisson's ratio, which is the ratio of the lateral deformation to that in the direction of stress. The meaning of this ratio as applied to a loaded column is the lateral deformation per unit of width divided by the longitudinal deformation per unit of length. In a slab supported on columns there is a similar condition of deformations caused by horizontal stresses at right angles to each other. But few tests, however, have been made to determine the value of Poisson's ratio and even the results which have been obtained vary considerably. Since Poisson's ratio has an important bearing on the value of the computed stresses, it has been pointed out that this is one objection to using the formulas of this method. Another objection lies in the fact that the analysis by Eddy assumes certain relations in both shear and moment curves which do not seem to be applicable to a plate supported over a considerable area at its center.

The third method of analysis mentioned is based on general deductions made from Grashof's analysis of the stresses in a homogeneous plate supported at rows of points forming squares. It has been pointed out that the expression found by Grashof is

[1] Year Book, Engr's. Soc., Univ. of Minn., 1899.

indeterminate for other than square panels and also that this method gives the maximum stress at the center of the cross band, when actual tests show this is at the support. We also find, as in the second method of analysis, that Poisson's ratio has an important bearing on the results. It would seem, then, from the above, that with our present knowledge of flat-slab design, the expressions derived from Grashof should not be considered of real practical value. Only the first two methods will receive consideration in this text. It must be understood that various results may be determined from each method, depending upon the assumptions made.

Tests and the results obtained by the first two methods of analysis indicate that the moment is greater over the support than at the center, and that the critical section is at or near the edge of the capital. When the column head is flared so that it becomes comparatively thin at its outer circumference, then a part of it is flexible and should be considered as part of the slab. For ordinary conditions, however, the edge of the capital may well be considered as the line of maximum bending moment.

It is common practice to maintain the same slab design for all floors. If this is done, the design should be based on a minimum size of column capital. The size of the column capital is clearly a function of the floor slab and not of the column, and should depend on the floor load and the panel size. Instead of arbitrarily assuming the size of capital, this dimension should preferably be based on computations for diagonal tension.

The shearing stresses in flat slabs seem exceedingly high, but there should be a distinction made between the so-called punching shear, and shear which measures diagonal tension. Punching shear may properly be figured along the periphery of the capital and this shear is readily borne by the concrete and steel if the structure is properly proportioned for diagonal tension. It should be noted in this connection that the Joint Committee recommends an allowable value in punching shear of 120 lb. per square inch for a 2000-lb. concrete.

Shear as measuring the tendency to diagonal tension may be computed at some distance out from the column capital. From a number of footing tests at the University of Illinois, Prof. A. N. Talbot concludes that, as an indication of the tendency to diagonal tension, the intensity of shearing stress should be computed on a section at a distance out from the pier or column equal to the

depth of the footing to the steel. The footings tested were of a uniform depth throughout and reinforced in a similar manner to the flat slab over a coum n head and it seems reasonable to apply Prof. Talbot's conclusion to the case of the flat slab. It may be pointed out that the portion of the floor slab inside the line of inflection is in effect an inverted footing with a uniform load over the base and a concentrated load all along the edge.

By referring to Fig. 67, the student will notice that the rods in a band do not all pass over the column head. It has been held that the beam strips in the outer part of a band serve only to carry load to other strips which they intersect and which pass over the capital. This contention has been disproved by tests made at the University of Illinois on wide beams, with the ratio of depth to width of about $\frac{1}{12}$, or somewhat in excess of the usual ratio of depth to steel to width of band in the flat slab. The beams were 36 in. wide, 4 ft. 10 in. long, and 3 in. deep to the steel. They were loaded across their full width at the ends and were supported at the third points, in some cases for their full width, in others for one-half their width, and in still others for one-fifth of their width. All the beams had ten $\frac{3}{8}$-in. round rods placed longitudinally, and the beams supported for only one-fifth their width had $\frac{1}{4}$-in. round rods 4 in. on centers crosswise of the beam. A study of the tests shows that there was only a very small falling off in the load carried when the beam was supported for only one-fifth its width, as compared to the load carried when the beam was supported its entire width. The deflection curves showed also that the steel at the edges took practically the same stress as did the rods in the beam supported for its full width. What amounts to the same result was observed in the footing tests above mentioned. Prof. Talbot concluded as a result of these footing tests that the width of the resisting section, as governing the stress in the steel, was composed of the width of the pier, plus the depth to the steel on each side of this, plus one-half of the remaining width of the footing. If this same conclusion is applied to a flat-slab floor, the width described includes all the steel in both the cross and diagonal bands in all ordinary designs. Thus it is evident that all the steel in the bands may be counted on to take its full resisting moment and not just the steel which passes over a column head. Actual tests bear out the same conclusion, the stress being found nearly constant across the entire band. The reinforcement in the footings previously mentioned was laid

sometimes in two bands and sometimes in four bands without any noticeable effect on the strength or stresses in the footing.

Tests have indicated that the moment at the center of a band for uniform loading is somewhat less than one-half that at the support. It may be possible, however, that, when only a single panel is loaded, the stress at the center reached this amount. It would seem, then, that this fraction may be considered a maximum for purposes of design.

Cracks were found in the field tests at a distance of about 2 in. on the average outside of the edge of the capital. This would seem to indicate that moments may safely be figured about a section at the edge of capital as giving a maximum condition.

The line of inflection has been found by test to lie between a square (having the column at its center) and its inscribed circle. Tests have also indicated that the diameter of the inscribed circle may be taken at 0,56 L; where L is the length of panel side. This diameter is the least dimension of the line of inflection, and occurs in the cross band direction. The maximum dimension of the line of inflection occurs along the diagonal band and was observed in the test to be about 0.64 L. It would seem from these figures that the line of inflection may, with sufficient accuracy, be considered to lie on a circle with a diameter of $\frac{0.56\,L+0.64\,L}{2}=0.60\,L$. The variation in this dimension would undoubtedly be small for the conditions found in practice, and, if columns are not permitted to have unduly small capitals, the value 0.60 L may safely be used. If the panel is not square, but nearly so, it is perhaps safe to consider L as the mean of the two panel dimensions.

The compression in the concrete at the under face of the slab surrounding the column is the absolutely vital and critical point in every flat-slab design. It is contended, and tests seem to indicate, that the tension in the concrete acts to reduce the tensile steel stresses over the supports very materially, even at high overloads. Arch action, which tests have clearly shown to exist, is another reason why steel stresses are low in a flat-slab floor. Both of these factors which reduce steel stresses tend to increase rather than decrease the compression in the concrete at the support. Using a method of design which does not call for more than one-half the amount of steel at the center of span that is required over support, it would seem from the results of tests that

a value of 750 lb. per square inch in the concrete at this point may be permitted for a 2000-lb. concrete.

When a spider is employed over the column head, the column or radial rods should be brought near the top of the slab at their extremity. It should be clear that hoops fastened to such rods cannot help in carrying tensile stress except as they lie above the neutral plane. It is the practice to-day to keep these rings and radials just as high as possible, consistent with allowing the slab rods to sag to the bottom toward the center of span. This was made necessary by the numerous slabs that showed circum-

Courtesy of Universal Portland Cement Co.

FIG. 71.—Larkin warehouse, Chicago, Ill. Note the depressed head design; also the floor sleepers and cinder-concrete fill.

ferential cracks that on investigation could be attributed in a larger measure to the steel being too low to take care of the negative bending moments than perhaps to any other cause.

Some buildings have recently been built with what is called a depressed head design—that is, the slab is deepened for a considerable distance around the column capital. Tests made by Mr. Arthur R. Lord on a building of this type—the Chicago warehouse of Larkin Co. (Fig. 71)—tend to show that the critical stress section may be expected at the edge of the depressed head, although, with the proportions used in the Larkin warehouse,

there was little difference between the stresses found at this point as compared with those found at the edge of the capital. The deflection readings, compared with those of other tests, show that a flat-slab floor with a substantial depressed head design is stiffer and stronger than the straight flat slab as ordinarily designed, with anywhere near the same amount of materials.

Cracks are liable to occur in flat-slab floors along rectangular lines between columns unless cross reinforcement of small rods is placed in the top of the slab. These cracks occur only along the cross-bands due to the fact that the span is shorter and the deflection is less than in the center of the slab.

Design of Interior Floor Bay

Design an interior bay for a flat-slab floor to support a live load of 200 lb. per square foot with columns spaced 16 ft. by 16 ft. on centers. Use working stresses recommended by the Joint Committee for a 2000-lb. concrete. Assume plain rods of medium steel.

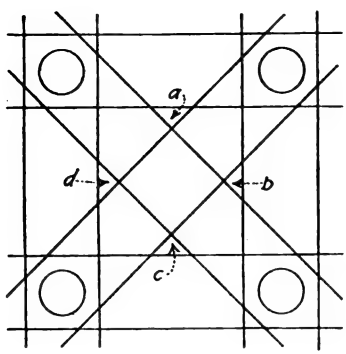


Fig. 72.

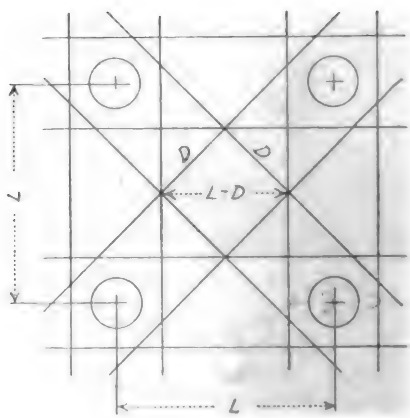


Fig. 73.

The width of each set of rods should be at least great enough so that no portion of the slab will be without reinforcement, as shown by the triangular areas a, b, c, d, in Fig. 72. The arrangement shown in Fig. 73 has no such gaps, and requires that

$$\sin 45^\circ = \frac{D}{L-D}$$

or

$$D = 0.414L$$

It would be better to take a value of D slightly greater than this in order to assure a slight overlap of the reinforcing rods, say $0.42L$.

Beam Method.—The diameter of the column capital will be assumed 54 in. For a continuous beam of span L, and carrying a total uniform load of amount W, the moment at the supports is $\frac{WL}{12}$, whereas the moment at midspan is one-half this amount, or $\frac{WL}{24}$. Assuming that each of the four sets of slab rods carries one-fourth the total load (which is not far from the truth since the load at the center of a panel is divided between two diagonal bands, whereas the load at the center of a cross band is taken by only one set of rods) the bending moment from which to determine the thickness of the slab and amount of reinforcement at the edge of column head becomes

$$\frac{\frac{W}{4}L}{12}, \text{ or } M = \frac{WL}{48}$$

For a width $D = 0.42L$, the moment per foot of width, say M_1, is therefore

$$M_1 = \frac{WL}{48(0.42L)} = \frac{W}{20} \text{ ft.-lb.}$$

By a preliminary calculation it is found that the floor slab will be about 8 in. thick, giving a dead load of 100 lb. per square foot. The total live and dead load is therefore 300 lb. per square foot. Consequently, $W = (16)^2(300) = 76{,}800$ lb. and

$$M_1 = \frac{76{,}800}{20} = 3840 \text{ ft.-lb.}$$

Thus,

$$d = \sqrt{\frac{M}{bK}} = \sqrt{\frac{3840}{133.8}} = 5.4 \text{ in.}$$

(The value of K is for $f_s = 16{,}000$ and $f_c = 750$. See Table 2 of Volume I.)

$$a_s = (0.0097)(12)(5.4) = 0.63 \text{ sq. in. per foot.}$$

Since the width of bands is assumed as $D = 0.42L = 6.72$ ft., say 6.75 ft., the total area of steel required for one radial system is $(0.63)(6.75) = 4.25$ sq. in As previously stated, the moment at

the center of the slab is one-half as great as at the supports. Thus, since the slab is to be of the same thickness throughout, only one-half as much steel is required in the bands at the center between columns as is required over the column tops. The required area of slab rods in one system is $\frac{4.25}{2} = 2.13$ sq. in., equivalent to 11 round rods, $\frac{1}{2}$-in. diameter, spaced 8 in. on centers. Over the columns, the rods should be lapped to double the area at this part of the floor.

The four sets of rods occur in layers where they cross the column top, and $d = 5.4$ in. is the distance from the extreme fiber in compression to the center of the nearest rod. The total thickness of slab should be $d + (3)(\frac{1}{2}) + \frac{3}{4}$ in. $= 7\frac{3}{4}$ in.

The thickness of the slab at the supports controls the thickness of slab throughout the floor. It is advisable in many cases to reduce the depth by using steel in both top and bottom at the supports, and by employing a rich concrete in the vicinity of the columns.

The area of concrete in shear at a distance out from the circumference of the column capital equal to the depth of slab to steel is $\pi(64.8)(5.4) = 1100$ sq. in. The total load on one column $= (16)(16)(300) = 76{,}800$ lb. Subtracting the load out to the section in question, which is approximately 6800 lb., the total shear to be considered is 70,000 lb. This gives a shearing stress on the section of $\frac{70{,}000}{1100} = 64$ lb. per square inch. If all the rods are properly bent up, it would seem that this shear, as measuring diagonal tension, is not excessive. This value, however, should be about the maximum allowed.

The floor has more than the necessary strength against punching shear.

Circular Plate Method.—The analytical processes involved in the analysis of stresses in homogeneous circular plates are very complex and only the results of such investigation will be considered here. The formulas adopted in preparing the diagrams of Figs. 74 and 75[1] apply to circular slabs free on their outer edge and clamped around the column.

Let M_1 = bending moment, per unit width of section, causing radial fiber-stress at any distance r.

[1] From "Principles of Reinforced Concrete Construction," by Turneaure and Maurer.

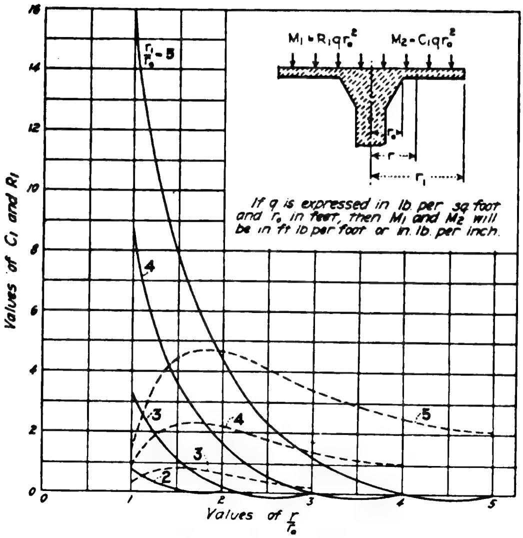

$M_1 = R_1 q r_o^2$
$M_2 = C_1 q r_o^2$
If q is expressed in lb. per sq. foot and r_o in feet, then M_1 and M_2 will be in ft. lb. per foot or in. lb. per inch.
Values of C_1 and R_1
Values of $\frac{r}{r_o}$

FIG. 74.

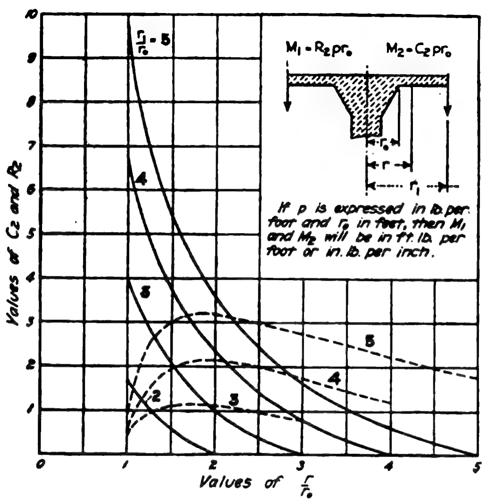

$M_1 = R_2 p r_o$
$M_2 = C_2 p r_o$
If p is expressed in lb. per foot and r_o in feet, then M_1 and M_2 will be in ft. lb. per foot or in. lb. per inch.
Values of C_2 and R_2
Values of $\frac{r}{r_o}$

FIG. 75.

M_2 = bending moment, per unit width of section, causing circumferential fiber-stress at any distance r.

q = load per unit area.

R_1 = ordinate to proper solid curve, Fig. 74.

C_1 = ordinate to proper dotted curve, Fig. 74.

p = load along circumference per unit length.

R_2 = ordinate to proper solid curve, Fig. 75.

C_2 = ordinate to proper dotted curve, Fig. 75.

Then

$$M_1 = R_1 q r_0^2 \text{ and } M_1 = R_2 p r_0$$
$$M_2 = C_1 q r_0^2 \text{ and } M_2 = C_2 p r_0$$

Fig. 74 gives the values of C_1 and R_1 for five different ratios of r_1 to r_0, or radius of plate to radius of column capital. For other ratios interpolations may be made. Similarly, Fig. 75 gives the values of C_2 and R_2. In each case the full lines give the coefficients for the radial bending moments (M_1) and the dotted lines those for the circumferential bending moments (M_2).

In deriving the above formulas the effect of lateral stresses has been taken into account, this being expressed by Poisson's ratio, which is the ratio of the lateral deformation to that in the direction of stress. The value of this ratio is taken at 0.1, which has been shown by experiments to be a fair value for concrete of 1:2:4 proportions.

It should be clear that the load assumed around the circumference of each plate, due to the load on the portion of the slab between these plates, may be determined per unit of length by dividing the load on the panel minus the load on one plate by the circumference of one plate.

Referring to Figs. 74 and 75, it is found that the radial maximum moments occur at the circumference of the enlarged column where $\frac{r}{r_0} = 1$. The circumferential moments at this point are a minimum and very much smaller than the radial moments, hence these latter only need be computed for maximum stress and the circumferential moments may be disregarded. The total maximum equals the sum of the radial moment at the edge of the column taken from Figs. 74 and 75 or from the following table for the proper value of $\frac{r_1}{r_0}$.

VALUES OF CONSTANTS R_1 AND R_2 BASED ON POISSON'S RATIO OF 0.1 AND WITH $\frac{r}{r_0}=1$

Values of $\frac{r_1}{r_0}$

	2.0	2.2	2.4	2.6	2.8	3.0	3.2	3.4	3.6	3.8	4.0	4.5	5.0
R_1..	0.72	1.08	1.52	2.04	2.66	3.37	4.17	5.06	6.05	7.15	8.35	11.77	15.99
R_2..	1.66	2.10	2.55	3.03	3.51	4.02	4.54	5.07	5.60	6.17	6.73	8.12	9.72

In the problem at hand, assuming the diameter of the column capital at 0.2 L, $r_1=\frac{(0.6)(16)}{2}=4.8$ ft., and $r_0=\frac{(0.2)(16)}{2}=1.6$ ft. The dead load will be assumed at 125 lb. per square foot, making the total load 325 lb. per square foot.

Area of slab (16×16) $=256$ sq. ft.
Area of circular plate $4.8^2\times 3.14$ $=$ 72 sq. ft.
184 sq. ft. area of slab outside of assumed circular plate.

$$\frac{(184)(325)}{(2)(3.14)(4.8)}=1990 \text{ circumferential unit loading in pounds per foot.}$$

Now, $\frac{r_1}{r_0}=\frac{4.8}{1.6}=3.0$, and from the above table

$$R_1=3.37,\ R_2=4.02$$

Employing formulas for maximum moments,

Total $M_1=(3.37)(325)(1.6)^2+(4.02)(1990)(1.6)=15{,}600$ in.-lb. per inch of circumference.

Some authorities advise placing only the two diagonal layers of steel in the top of the slab for tension and the two rectangular layers in the bottom of the slab to aid in compression. In the present case 2 per cent of steel will be placed diagonally in two layers at the top and 1 per cent of steel rectangularly in two layers at the bottom. It will be assumed that this amount of steel is economical and the resulting thickness of slab satisfactory. Using Table 11 of Volume I, assuming $\frac{d'}{d}=.15$

$$d=\sqrt{\frac{M}{f_cLb}}=\sqrt{\frac{15600}{(750)(0.285)(1)}}=8.55 \text{ in.}$$

We will take $d=8.5$ in. and a total depth of 10 in. For this depth the dead load is $\frac{10}{12}(150)=125$ lb. per square foot, the same as assumed. Also $\frac{d'}{d}=\frac{1.5}{10.0}=.15$, the same as assumed. The corresponding stress in the tensile steel is

$$f_s=\frac{15600}{(0.017)(8.5)^2}=12{,}700 \text{ lb. per square inch.}$$

Since an increase of reinforcement for a short length over the columns decreases the thickness of entire slab, several trials should be made in any given design to determine the most economical relation of the amount of steel and concrete. A rich concrete mix, allowing a higher stress in the concrete, would undoubtedly be of advantage here. It would be well to limit the stress in the concrete to 800 lb. per square inch for any mix.

The total amount of steel required in the top of slab over column is $a_s=bdp=(2)(3.14)(1.6)(12)(8.5)(0.02)=20.5$ sq. in. Each layer is effective on both sides of column and hence must have $\frac{20.5}{4}=5{,}1$ sq. in. of steel. Since the width of band is 6.75 ft., ½-in. round rods spaced 3 in. apart will give sufficient reinforcement. At the bottom of slab at column, the ½-in. round rods will be placed 6 in. on centers.

If we let l_1 equal the distance between lines of inflection, it would appear amply safe to adopt a value of $M=\frac{wl_1^2}{12}$ at the center of the cross-bands. For the diagonal reinforcement, the steel runs in two directions and $M=\frac{wl_1^2}{24}$ may be safely employed. Tests indicate that the moment is even less than this.

The diagonal distance between the circles of inflections is $22.6-9.6=13$ ft., and the bending moment in the middle of the panels, using the conservative formula $M=\frac{wl_1^2}{24}$, is

$$M=\frac{(325)(13)^2(12)}{24}=27{,}400 \text{ in.-lb. per foot of width.}$$

Considering 1½ in. of concrete below the center of steel, the effective thickness of the slab is 8.5 in.

$$K=\frac{M}{bd^2}=\frac{27400}{(12)(8.5)^2}=31.6$$

In Table 3 for $K=31.6$, $p=0.0022$, and thus only 0.22 per cent of

steel in each diagonal direction will be necessary, or ½-in. round rods 10 in. apart. As a matter of convenience, the rods would naturally be placed 9 in. apart. The spacing of the rods in the rectangular bands may be determined in like manner.

The strength of the slab against diagonal tension may be determined in the same manner as in the beam method.

Problem

7. Design an interior bay for a flat-slab floor to support a live load of 150 lb. per square foot with the columns spaced 18 ft. by 18 ft. on centers. Design by the two methods described in the text. Use working stresses recommended by the Joint Committee for a 2000-lb. concrete and medium steel.

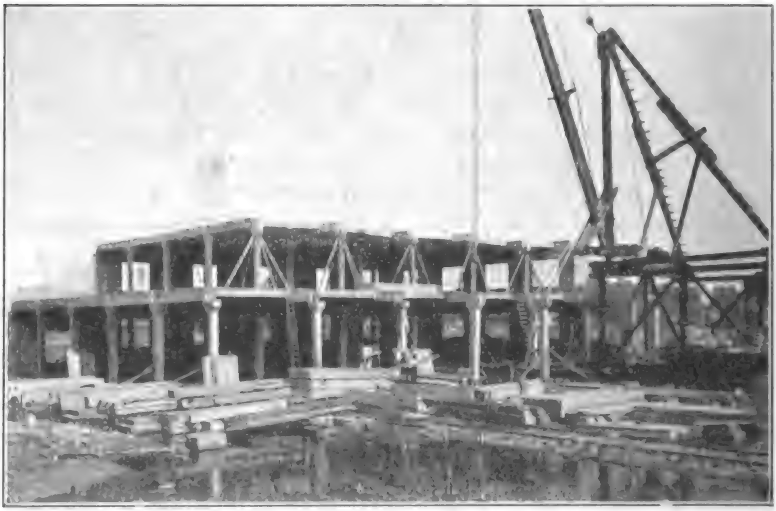
19. Unit Construction.—In the *unit* system of reinforced-concrete construction, all members (except the floor slabs in the Ransome System) are cast in forms on the ground and set in place, when hard, by a derrick (Figs. 76 and 77). The difficulty of securing rigid connections between girders, beams, and columns has been the most serious defect in this type of construction, but much has recently been accomplished in this respect.

Economy can be the only reason for employing the *unit* type of construction, as it is not any stronger or better looking than the monolithic construction; in fact, in these respects its highest expectation can be but equality. In the matter of cost, however, considering ordinary cases, it seems to have the better of the argument for three main reasons: (1) the greatly reduced amount of falsework required as compared with the monolithic type, (2) the reduction in the number of men required for the work of construction, and (3) the chance to carry on the work under cover in all kinds of weather.

With the exception of structures built by the Ransome Unit System, each member in a *unit* floor is designed as an individual carrying element, quite separate from the superimposed slab, and receiving no assistance therefrom. This necessitates the use of extra deep beams or the use of T-beam sections in order to get the required compressive strength. With the exception of the Ransome System just noted, the units are tied together by virtue of bars projecting into pockets or open spaces in which concrete is poured.

9

The Ransome Unit System differs principally from the other unit systems in that the slab is poured in place after the unit beams, girders, and columns have been erected. If the tests described in Art. 28 are any criterion, it is possible to so unite the slab and beam that they may be considered as working as a unit—thus making possible, where continuity is not considered, just as economical a design from the standpoint of material as in



Courtesy of Universal Portland Cement Co.

FIG. 76.—Unit construction. Sturges & Burn Mfg. Co.'s building, Bellwood, Ill.

the monolithic type. In this system the slab serves to tie the entire building together—a feature lacking in the other unit systems.

It should be quite evident that the greatest advantage obtained from the use of separately molded members occurs when a large number of the same size of beams, columns, and girders are to be employed. The same forms can then be used over and over again. From this it follows that the unit type of construction is not likely to have universal application due to the multiplicity of shapes in complex structures. Study of designs, however, should in most cases reduce the number of shapes to a workable minimum. Restriction in sizes of discontinuous members is also likely to be a controlling factor and, under some conditions,

narrow the application of this system to certain layouts governed by the capacities of the handling apparatus.

Shrinkage cracks do not usually occur in buildings of unit construction since all the shrinkage has taken place in the individual units before their incorporation into the structure. It is probable that unit methods will advance concrete construction to a par with steel construction in that every element may

Courtesy of Unit Construction Co.

FIG. 77.—St. Louis Lead & Iron Works, St. Louis, Mo. Constructed by the unit method.

be inspected and approved before it is placed, or, if desired, a given percentage of the units may be tested before erection.

The principles of unit construction are well illustrated in recent types of structures built by the Unit Construction Company of St. Louis (*Unit-Bilt* System), and the Ransome Engineering Co. of New York—types which will now be described. It may as well be said here that the Unit Construction Company has very recently acquired the patents covering the Ransome Unit System and thus both systems are now controlled by one company.

Figs. 78 to 81, inclusive, and Plate X show the principle of construction of the *Unit-Bilt* System. The columns are provided with brackets for the support of the girders, and the girders are

set on a mortar bed at a distance back from the center of column sufficient to allow the column rods to overlap. The girder rods to take negative moment over supports project into the space over the columns—the girders being cut back so as to give the necessary length for embedment (Fig. 79 and Plate X). The ribbed slabs rest on shelves or ledges on the sides of the girder,

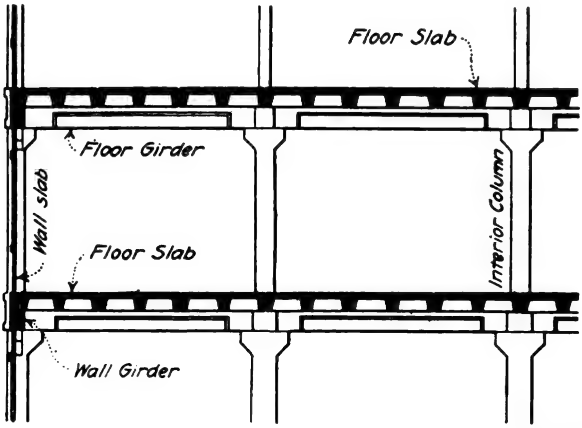


Fig. 78.—Part section of building on the "Unit System" of reinforced-concrete construction.

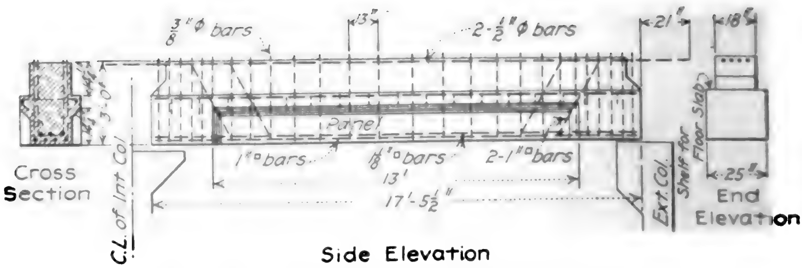


Fig. 79.—Typical girders for "Unit System."

with the top of slab above the top of girder so as to permit the slab reinforcing-rods to project into the space thus formed and provide for negative tensile stresses. The outer face of each side rib of slab has a groove, so that the two grooves of adjacent slabs will form a key for the grout filling. When the spaces over columns and between the floor slabs are filled with concrete, a very rigid construction results. No attempt is made in the field to

obtain a complete bond between the filled concrete and the concrete of the separately molded members, since the filled concrete is used either in direct shear or in compression, or as a means for bonding the projecting rods in the space between the units.

The column brackets are designed for shearing and bending, and proportioned to the bearing aiea required by the girder. The girders are figured as rectangular in section, although after

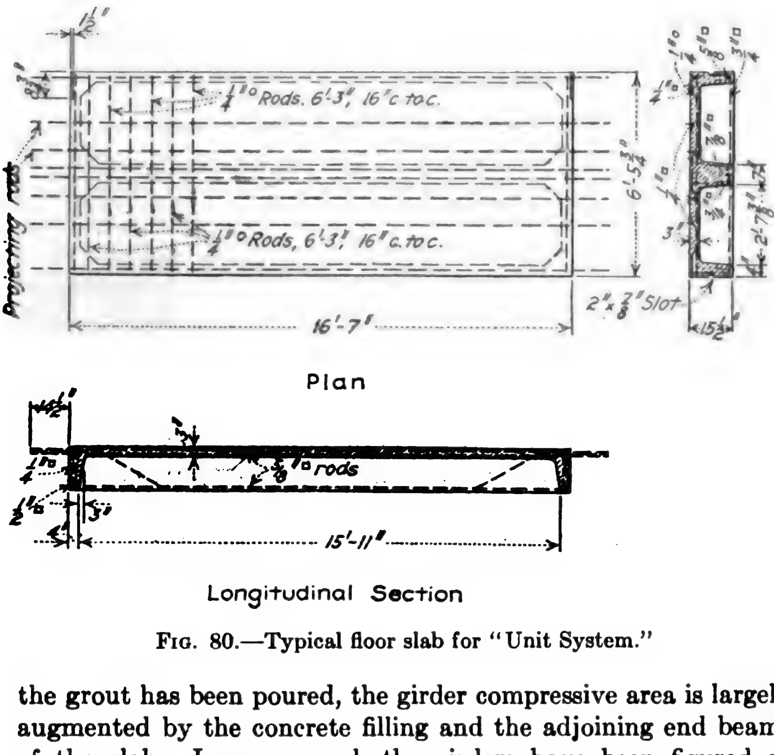


FIG. 80.—Typical floor slab for "Unit System."

the grout has been poured, the girder compressive area is largely augmented by the concrete filling and the adjoining end beams of the slab. In some work the girders have been figured as simply supported. Where continuous action has been considered, the girder is figured continuous only for the live load, since this load comes upon the girder after the connections are grouted in.

The main details of the Ransome Unit System are shown in Plate XI. The usual reinforcement is placed near the sides of the columns and, in addition, a longitudinal rod is inserted in a central cored hole extending lengthwise through the column. The grouting of the column is done from the top after the setting

of the girders and beams. The cored hole is enlarged and flared out at the bottom in order to insure an even bed for the column. The main column reinforcement is not continuous from story to story and the caps and bases of the columns are enlarged so that, at these points, the concrete alone will be able to transfer the weights.

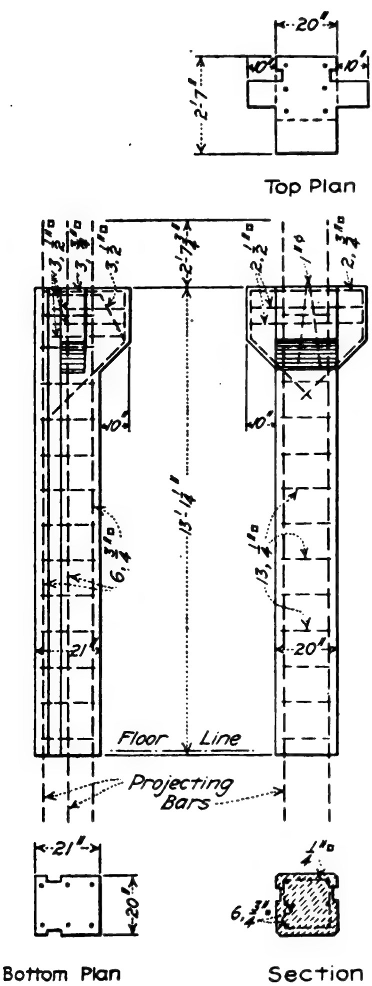


FIG. 81.—Details of column construction, "Unit System."

The main girders are placed on top of the columns—the ends of each girder being enlarged so as to almost cover the cap of the column. The beams are made with dove-tailed ends and fit into pockets in the girders (Plate XI). In the design of this system, the vertical stirrups must be arranged so as to thoroughly bind the slab and beam, and make these members act as a unit. The beams and girders are usually cast of a depth equal to the distance from the bottom to the neutral axis only (Plate XI). The slab forms are erected between the beams (which are usually spaced about 4 ft. on centers) and rest upon ledgers bolted to the sides of the beam. The beam and girders are joined to the slab with a beveled joint, giving a slight draft to the forms, thus affording considerable concrete around the connecting rods.

There is no shoring placed under the floor during construction so that the beam and girder units should be designed strong enough to carry their own weight plus that of forms and wet concrete slabs, in addition to an allowance for impact from buckets and so forth. The slab panels may be removed in a much

PLATE X

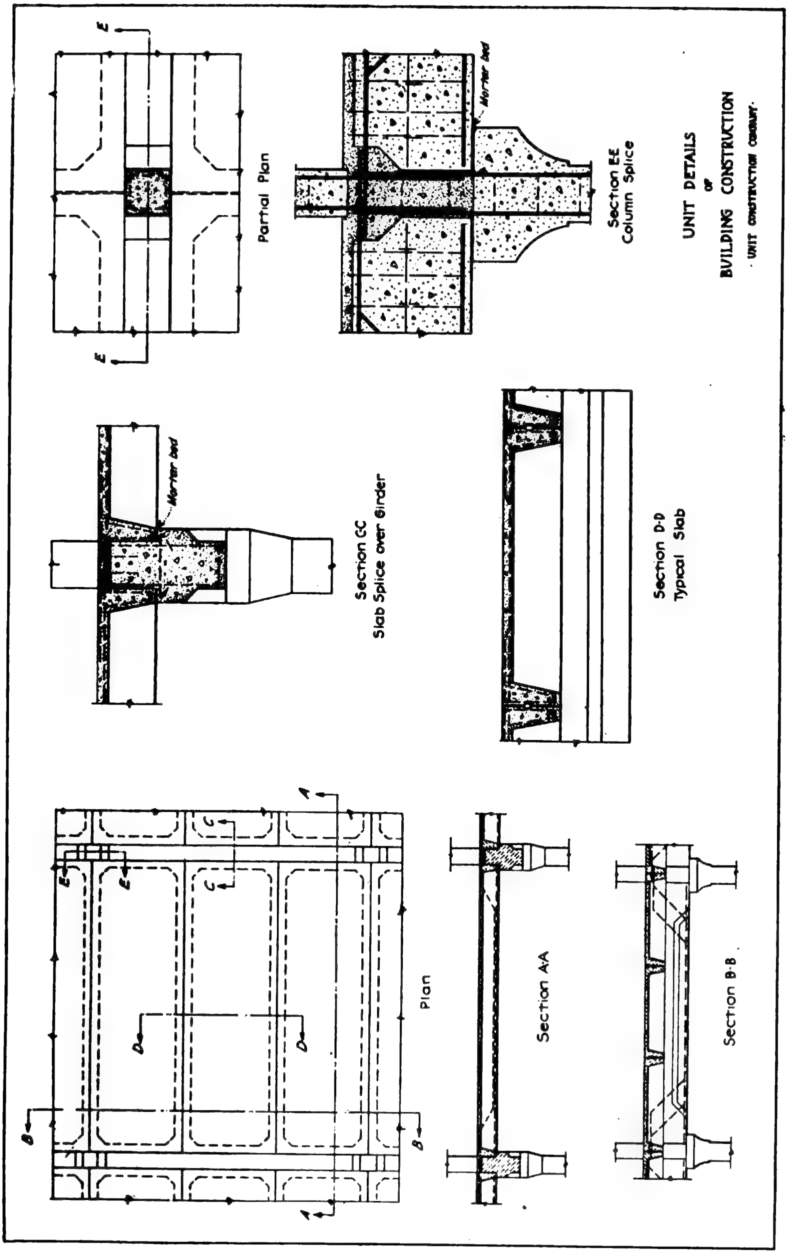
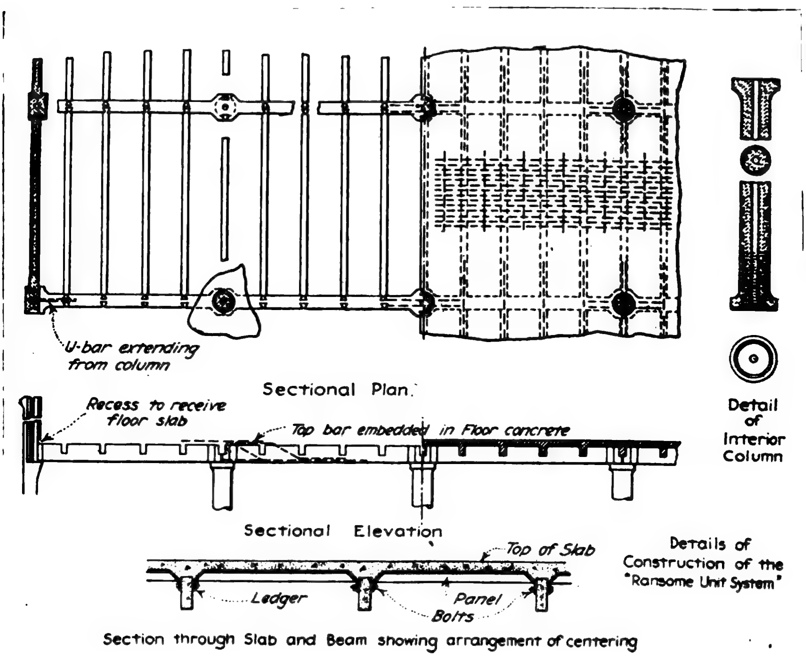

Partial Plan
Section EE
Column Splice
Mortar bed
Section CC
Slab Splice over Girder
Section D-D
Typical Slab
Plan
Section A-A
Section B-B
UNIT DETAILS
OF
BUILDING CONSTRUCTION
UNIT CONSTRUCTION COMPANY

PLATE XI


U-bar extending from column
Sectional Plan.
Recess to receive floor slab
Top bar embedded in Floor concrete
Sectional Elevation
Top of Slab
Ledger
Panel Bolts
Section through Slab and Beam showing arrangement of centering
Detail of Interior Column
Details of Construction of the "Ransome Unit System"

shorter time than would be permitted on monolithic construction, since the previously molded beams and girders take care of the entire load up to the time when the full live load is brought on the floors.

The beams in some buildings have been connected by means of tie bars as shown in Fig. 82. At about the quarter points and in the top of each beam and girder a hole was cast, extending down into the beam about 3 in. From this to the end of beam a slot was formed. With two beams end to end, a rod with a right-angle bend at each end could then be slipped in place before the slab was poured. In the more recent developments of the system

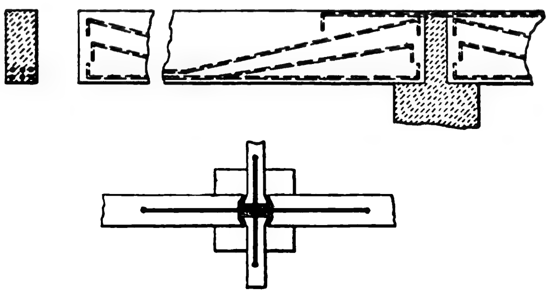


FIG. 82.

(Plate XI), the ends of the reinforcing rods project above the tops of the beam and girder units and a loose rod is inserted in the slab to provide for negative moment. The beams and girders are figured as simply supported although, with proper design, the continuity might probably be taken advantage of to some extent.

20. Steel Frame Construction with Concrete Slabs.—The steel skeleton consists of columns, girders and cross-beams—the same arrangement as in the monolithic beam and girder construction—the beams usually being spaced about 6 ft. on centers. The many styles of this type of construction differ from each other in the form of slab steel used, the position of the concrete relative to the beam, and in the use of curved or flat slabs.

I-beams comletely enclosed in concrete should be wrapped with wire, wire mesh, or metal lath, to prevent the concrete below the bottom flange from cracking and dropping off. The material used should be of sufficient size to render efficient service even though it should, by some accident, become corroded.

It should be wrapped securely to the I-beam flange, and, at the same time, sufficient space should be left to effect a concrete clinch between the wrapping material and the beam.

Fig. 83 shows the floor slab placed directly on the tops of the

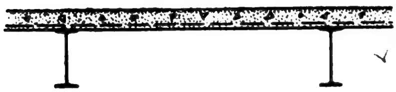


Fig. 83.

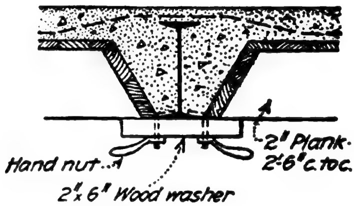


Fig. 84.

steel beams. The reinforcement of the slab may be either small rods, a wire fabric, or a sheet reinforcement. The slab as shown must be calculated as a simple beam, since reinforcement is not provided for negative moment over the supports. The method of

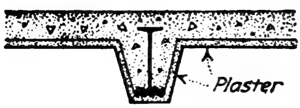


Fig. 85.

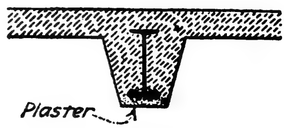


Fig. 86.

figuring such slabs and a discussion relating to the reinforcement for negative moment is given in Chapter V of Volume I.

Fig. 84 is a very common form of concrete floor supported by steel girders. The form for constructing same is also shown. The haunches of the slab are carried down to the lower flange of

Fig. 87.

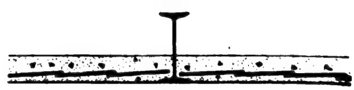


Fig. 88.

the I-beam, the under surface of which may be covered with metal lathing and plaster for fire protection. (See Figs. 85 and 86.) The I-beam is sometimes entirely encased in the concrete as shown in Fig. 91, but it is difficult to place the material under the lower flange.

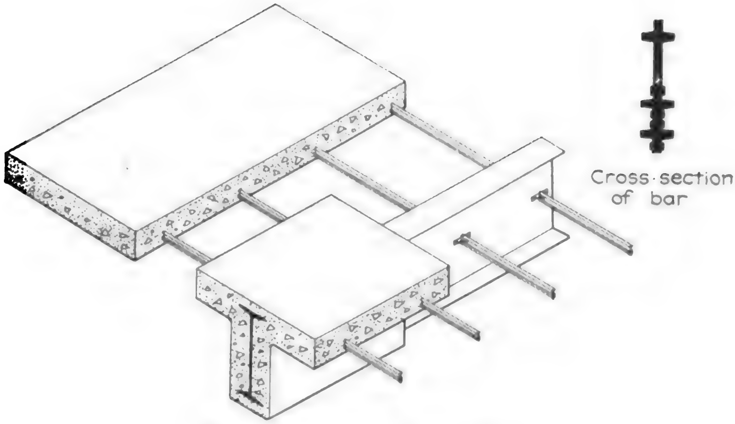


FIG. 89.—Columbian slab floor.

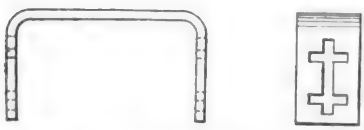


FIG. 90.—Hanger for Columbian bars

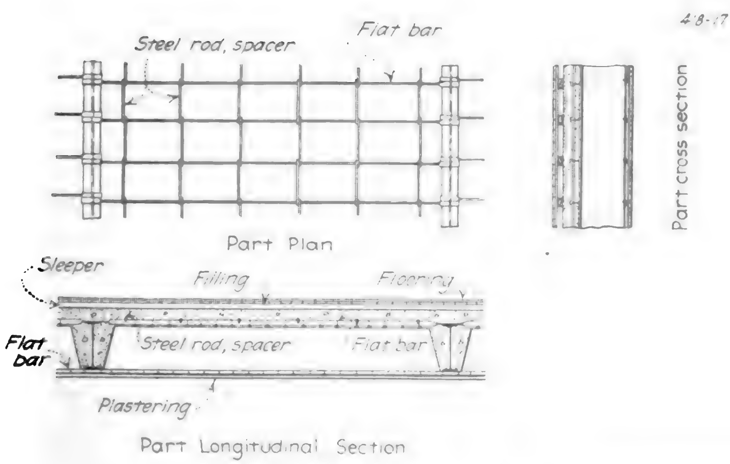


FIG. 91.—Roebling flat-slab floor.

Concrete floors are sometimes laid as continuous slabs with only the upper part of the I-beams in the concrete, and sometimes the slab is so located with reference to the I-beams that the metal is placed between the beams instead of running over them, as in Figs. 87 and 88. Still another type of floor consists of arches sprung between the lower flanges of the I-beams and filled to the floor level with cinders.

In Fig. 89 is shown the Columbian slab floor. The reinforcing bars are suspended from the top flange of the I-beam by a hanger, as shown in Fig. 90, or they are riveted to the web of the beam as shown in Fig. 89.

The Roebling slab floor is of many types, a common form being shown in Fig. 91. The reinforcement is flat bars, which are bent

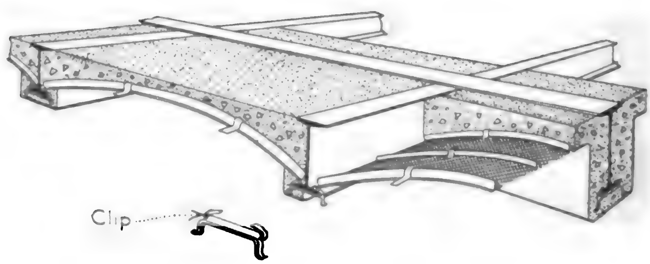


FIG. 92.—Roebling segmental concrete arch floor.

at the beams so as to connect with the flange as shown. Spacers, which supply the place of distributing rods, are fitted into slots in the bars. A 16-ft. span may be constructed with this type of floor. A flat ceiling is obtained by suspending metal lathing from beam to beam and plastering. A Roebling segmental concrete arch floor is shown in Fig. 92.

21. Floor Surfaces.—A concrete floor usually has a mortar or granolithic finish as wearing surface. Such a surface if not allowed to set rapidly is hard and practically impervious to water.

The usual proportions for granolithic finish are one part Portland cement, one part sand, and one part crushed stone which passes through a ¼-in. mesh screen. This mortar surfacing is laid over the concrete slab and troweled to a hard finish. If placed before the concrete below has set, it may be from ¾ in. to 1 in. thick, but where the concrete of the slab becomes old before the granolithic finish is laid, the thickness should be at least 2 in.

Granolithic finish is screeded to grade with a straight edge, smoothed with a wooden float, and finished with a steel trowel. It is often marked off into blocks, or sections, of suitable size by shallow grooves. The object in dividing the surface into panels is purely ornamental, since reinforcement against shrinkage cracks is provided in the lower portion of the concrete slab.

Some engineers prefer to lay a cinder concrete base, not less than 2 in. thick, before placing the cement finish. The advantage of this method is that should there be any reason for removing a section of the finished floor, it can be accomplished without injury to the reinforced-concrete slab. On account of the porosity of the cinder concrete it is difficult, however, to obtain a satisfactory cement finish if the same is placed before the cinder concrete base has set.

It is quite often necessary in warehouses and factories to protect concrete floors against heavy trucking. Discussing the methods which have been employed by the Aberthaw Construction Co., Mr. M. C. Tuttle writes as follows in the *Cement Age*, issue of February, 1912:

"We have used several methods for this, particularly in paper mills where the wet stock is handled in big boxes which are supported by small iron wheels. Here we have used cast-iron plates which were bedded in the concrete, and have also used a rack made of flat bars of steel with washers between them; in general, like a rack for a head gate in a canal. The edges of the flat bars take up the wear of the trucking and have proved quite satisfactory. This is probably the cheapest form of steel wearing surface which we have used."

If a wood floor is desired, 2×3-in. or 2×4-in. sleepers are usually laid on top of the rough concrete slab, and cinder concrete or stone concrete poor in cement is run between these nailing strips. For ordinary cases, ⅞-in. to 1¼-in. maple flooring nailed to 2×3-in. sleepers, 16 in. apart, is found satisfactory. The proportions of cinder concrete specified vary considerably—average proportions, 1: 3: 6. The sides of sleepers are usually beveled, but this does not prevent them from becoming loose when the wood shrinks. The best method of holding the sleepers in place is to drive 40d nails in the sides of the sleepers at intervals of three feet, on alternate sides. These nails key with the concrete and prevent any movement of the sleeper.

There has been much dissatisfaction with the above method of constructing wooden floor surfaces on account of the liability of

the sleepers to decay by dry rot. To avoid decay the sleepers should be well seasoned, and care should be taken to permit no moisture to reach them even after the top flooring is in place. Sufficient time should be allowed the concrete to dry out before the finished flooring is laid. Dry rot is a fungus disease of wood that thrives in the presence of moisture and absence of circulating air. It makes little difference whether the moisture is the sap of the green wood, or water that soaks into the wood after it has been dried.

Sleepers are sometimes laid in dry cinder fill. In such construction the chance of dry rot is probably small, since there is some opportunity for air circulation through the porous cinders. Cinders would also tend to absorb the moisture.

A sand-and-tar base for plank flooring on reinforced-concrete floor slabs has been employed to some extent to avoid any danger from dry rot. A layer of sand mixed with coal tar is spread about 1¼in. thick on the floor slab and leveled while still warm and soft with a straight-edge. On this layer is then placed first a layer of 2-in. plank, then ⅞-in. rough pine board, and finally a wearing surface of ⅞-in. to 1¼-in. maple. The different layers of planking should be placed in different directions.

The following construction has been used in school-house floors with satisfactory results: At the completion of the floor slab, a 1-in. footing of sand is placed on top of the same and brought to a true level by screeding. On top of the sand are placed 1¼-in. boards laid diagonally and nailed together at the edges. The form lumber used for centering is generally employed for this work. On top of the 1¼-in. boards is then placed building paper, which in turn is covered by the finished flooring.

In hotels and similar establishments, linoleum or carpeting over a fairly smooth cement base seems to give satisfaction. The following discussion by E. J. Ruxton[1] in *Concrete Cement Age* for October, 1912, should be of interest:

"In regard to the sawdust concrete floors laid at Springfield Public Library, the conditions and results in this work are approximately as follows:

"Several preliminary samples were made up of cement, sand and sawdust in the following proportions: 1:2:2, 1:2:3, 1:4:½, 1:3½:½, 1:3:½, and 1:2½:½.

[1]Member of the firm of Birnie, Adams & Ruxton Construction Co., Springfield, Mass.

"The cement used was both Lehigh and Iron Clad cement on which no previous tests had been made. The sand was a clean, sharp sand secured locally, and the sawdust was that obtained from a local box factory, consisting of the sawdust from native pine wood, most of which probably was green wood. These samples were made in a box 1 ft. square and 3 in. high, having no bottom, and resting directly on reinforced-concrete floors that had been installed. After allowing these to set about 2 weeks, it was found that of all the samples made, the only two which seemed at all possible to use were the mixtures 1:2:1 and 1:2:2, although the architect had specified a 1:2:3 mixture.

"In laying the floors a 2-in. base course of cinder concrete, mixed in proportions of 1:3:5 was first placed on the reinforced-concrete floor, and on this was placed a 1-in. finish course of 1:2:2 saw-dust mixture. The first day's run we installed about 265 sq. yd., but because of rainy weather we had to discontinue operations for about 4 days. At the end of this time, it was seen that this original 265 sq. yd. had not hardened sufficiently to sustain any weight, and also showed no holding power for nails, which were to be driven into it finally. It was then decided to change this top mixture to 1:2:1, and when the operations were continued this mixture was used, which gave better results.

"However, it was seen that this top mixture might possibly be made still better by changing it to the proportions of 1:2:$\frac{3}{4}$, this latter mixture being used for the remainder of the 5000 sq. yd. installed.

"The writer would state that in laying this floor it was not the intention to construct a floor which would stand the ordinary wear on its surface, as it was the intention to produce a surface on which a cork carpet could be laid, providing strong holding power for the nails and at the same time producing a softer surface for a carpet than the ordinary 1:2 cement-sand mixture would produce. It was found that with the mixture of 1:2:$\frac{1}{2}$ it was practically impossible to drive nails in it, but in the mixture 1:2:$\frac{3}{4}$ no difficulty was noticed in driving the nails and at the same time they had good holding power.

"In working this top finish, it was the intention to place it not later than 50 minutes after the bottom cinder concrete was placed. This result, however, could not always be obtained because of rainy weather, so that in the event of a longer time having elapsed between the placing of the finish coat on the bottom course, a wash of cream cement and water was first applied, before the top or finish coat was placed.

"In regard to the troweling or finishing of this coat, 1 in. × 2 in. screeds were placed on the bottom or cinder concrete, and leveled, and the top course placed and screeded down with a straight-edge having a steel face. The object of this was to secure a smooth surface requiring no troweling, as this top course set very slowly, thus delaying any opportunity for troweling. Wherever there were any rather rough spots these were usually rubbed down the next morning, the top course

having set sufficiently to sustain the weight of a man and allow troweling.

"For the troweling, we used an ordinary wood float instead of a steel trowel. This gave a smooth enough surface for laying a cork carpet, and prevented unnecessary travel over the green floor to secure this. About two weeks after this surface of sawdust concrete was laid, the interior decorating and plastering of the library building was done. This necessitated the floor carrying heavy horses for platforms, and heavy traffic, and at the end of the plastering period it was necessary to remove all surplus plastering from the floor. Even with this hard usage very little patching had to be done before the cork carpet could be laid.

"It will be two years this fall since this floor was installed, but as it is covered with the cork carpet, we have no way of telling in just what condition it may be. We have, however, heard no complaint in regard to the carpet becoming loose or the nails pulling out."

Flat tiles are sometimes employed as a floor surface in reinforced-concrete construction. The most durable and sanitary tile is the vitreous-clay tile. The *ceramic* tile is perhaps the most often used and is a vitreous-clay tile manufactured in square, hexagonal, and round shapes. The squares range in size from ½ in. to ¾ in. and the hexagonal shapes from ¾ in. to 1 in. Tiles are laid some little time after the floor slab is poured and, if set in Portland-cement mortar, should be embedded in a layer not less than 2 to 2½ in. thick in order to prevent *curling* of the tile due to shrinkage of the base. There are now on the market, however, some patented bases which seem satisfactory and which will allow a bedding thickness of only 1 in. over and above the thickness of the floor slab proper.

22. Loading.—The following loading for floors, extracted from the Boston building laws, represents first-class modern practice:

"All new or renewed floors shall be so constructed as to carry safely the weight to which the proposed use of the building will subject them, and every permit granted shall state for what purpose the building is designed to be used; but the least capacity per superficial square foot, exclusive of materials, shall be:

"For floors of dwellings and for apartment floors of apartment and public hotels, 50 lb.

"For office floors and for public rooms of apartment and public hotels, 100 lb.

"For floors of retail stores and public buildings, except schoolhouses, 125 lb.

"For floors of schoolhouses, other than floors of assembly rooms, 80 lb., and for floors of assembly rooms, 125 lb.

"For floors of drill rooms, dance halls and riding schools, 200 lb.

"For floors of warehouses and mercantile buildings, at least 250 lb.

"The loads for floors not included in this classification shall be determined by the Commissioner, subject to appeal, as provided by law.

"The full floor load specified in this section shall be included in proportioning all parts of buildings designed for dwellings, hotels, schoolhouses, warehouses, or for heavy mercantile and manufacturing purposes. In other buildings, however, certain reductions may be allowed as follows: In girders carrying more than 100 sq. ft. of floor, the live load may be reduced by 10 per cent. In columns, piers, walls, and other parts carrying two floors, a reduction of 15 per cent of the total live load may be made; where three floors are carried, the total live load may be reduced by 20 per cent; four floors, 25 per cent; five floors, 30 per cent; six floors, 35 per cent; seven floors, 40 per cent; eight floors, 45 per cent; nine or more floors, 50 per cent."

In a factory or warehouse it is often possible to calculate accurately the maximum weight which a floor will be called upon to carry. For the very heaviest loading the problem is frequently the simplest since such loading is apt to be due to certain definite weights, such as the weight of merchandise. The loading, however, must often be assumed without actual computation, and the dynamic effect of the live load will often require consideration. Mr. Sanford E. Thompson in "Reinforced Concrete in Factory Construction," published by The Atlas Portland Cement Co., suggests the following values as a guide to the selection of floor loads:

Office floors	100 lb. per square foot.
Light running machinery	200 lb. per square foot.
Medium heavy machinery	200 lb. per square foot.
Heavy machinery	250 lb. per square foot.
Storage of parts of finished products, depending upon actual calculated loads	150 to 500 lb. per square foot.

Slabs and beams in floor construction should always be calculated for full loading, but girders and columns are sometimes designed for a slightly smaller load if the loading at any given time occurs over only a part of the floor.

The working stresses usually employed, and those recommended by the Joint Committee (see *Appendix*) are intended to apply to static loads. Proper allowance for the dynamic effect of the

live load should be taken into account by adding the desired amount to the live load to produce an equivalent static load before applying the unit stresses in proportioning parts. An allowance for impact will be necessary only in special cases, as in the case of floors supporting heavy machinery. The amount to add to the live load because of impact will vary all the way from 25 per cent to 100 per cent depending upon the proportion of the specified live load which may be subject to motion.

The dead load of any floor may be estimated from the following approximate data—weights are per square foot of floor surface:

Wooden wearing surfaces, 4 to 6 lb.
Screeds or nailing strips, 2 lb.
Cinder-concrete filling (2 in. thick), 15 lb.
Hollow tile, see Art. 17.
Plaster, 5 lb.
Suspended ceiling, 10 lb.
Cinders, 7 lb.

23. Economical Considerations.—The size of floor bays depends upon the loading, the uses to which the building is to be put, and the size and shape of the ground area. In a large building it is frequently worth while to make several comparative estimates with different layouts of the beams, girders and columns, so as to obtain the most economical arrangement under the given conditions.

The cost of changing forms to meet changes in sizes of beams and columns in different stories must be kept constantly in mind. It is more economical to vary the depths of beams from floor to floor than the widths, on account of the slab panels; and to vary square columns in one dimension rather than in two, both on account of the columns themselves and the beams which frame into them. It is not advisable to change the size of members from floor to floor for small differences in computed dimensions; nor is it advisable on any floor, if simplicity in field work is desired, to make slight changes in the sizes of beams and in the thicknesses and reinforcement of the slabs, unless such variations occur over large areas. When slabs of considerable variation in span alternate, it is better to vary the thickness and keep the reinforcement the same, than to vary the reinforcement and use the same thickness of slab. Although slight differences in dimensions are not desirable, the designs made should be considered carefully in every detail.

24. Small Floor Openings.—The methods of arranging slab reinforcement around small openings, such as for chimneys and ventilators, needs some consideration. Both a wrong and a right method are shown in Fig. 93. The openings are shown against a wall, and the floor slab is reinforced in only one direction.

The method shown of placing the reinforcing rods parallel to the opening makes a neat looking job, but it should be evident that no proper provision is necessarily made to carry the load which would ordinarily come on the wall at the openings. The right method shown in less artistic, but, as a general rule, is

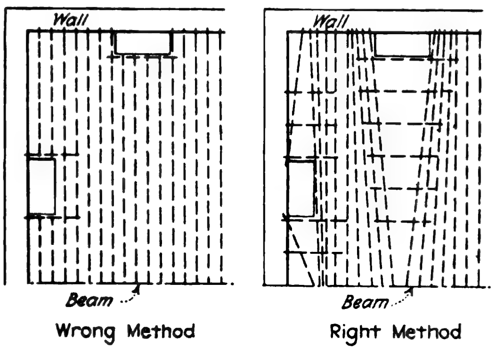


FIG. 93.—Method of placing reinforcement around small floor openings.

vastly more efficient. The cross rods serve only to reinforce the slab locally. All such designs should be carefully studied to make sure that the required strength is secured at all points. Slab rods should never be curved horizontally to dodge floor openings as a curved rod will tend to rotate.

Wrought-iron and galvanized-iron sleeves are built into the construction work for all steam, return, sprinkler, sewer, gas, and similar pipes. All floor sleeves should be flush with the ceiling line and should extend about 2 in. above the floor line. Pipe-risers should not be allowed to come up through columns as repairs and alterations are difficult, if not impossible, under such an arrangement; electric conduits, however, form an exception to this rule. Special shafts with fire-proof walls are sometimes used for plumbing and vent pipes, and this practice has much to commend it since a floor to be a perfect fire cut-off should be solid from wall to wall, with stairways, elevators, and all openings enclosed in vertical fire-proof walls.

25. Provision for the Attachment of Shaft-hangers and Sprinkler Pipes.—There are many different methods of attaching shafting, sprinkler pipes, and machinery to the under side of a

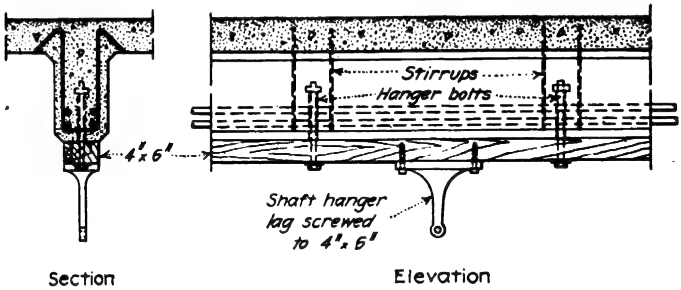


Fig. 94.

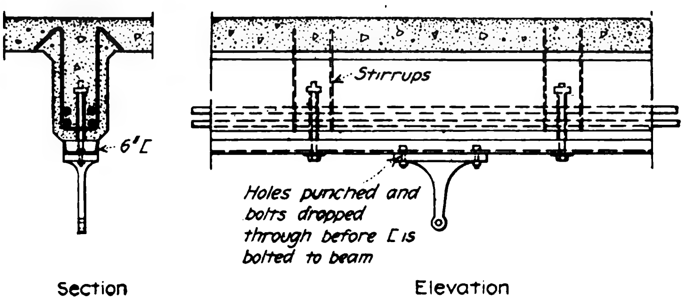


Fig. 95.

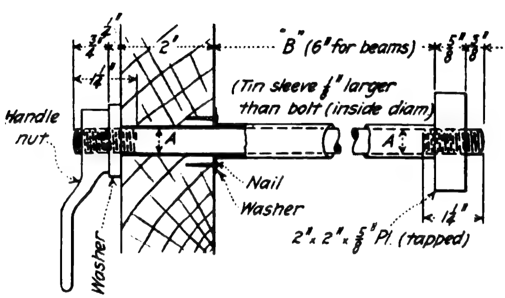


Fig. 96.—Detail of socket and bolt.

concrete floor. The method adopted should be as flexible as possible in order to accommodate future changes in the line of shafting, additions, or improved machinery. All bolts and

sockets should be placed before the concrete is run, as drilling is expensive and reinforcing bars are liable to be encountered, which would cause more or less difficulty and might lead to serious trouble if cut off.

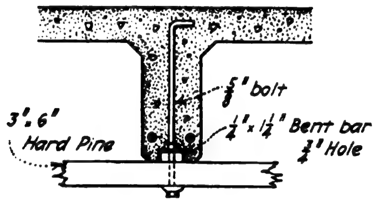


Fig. 97.

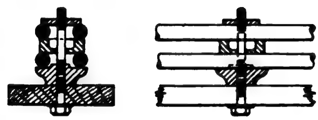


Fig. 98.

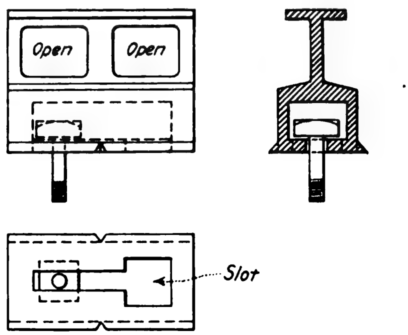


Fig. 99.

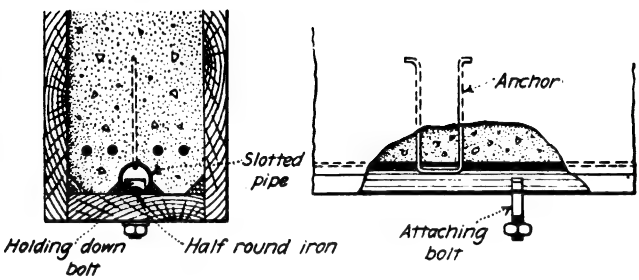


Fig. 100.

A convenient method is to place suitable bolts at intervals, usually 4 ft. on centers, with their threaded ends projecting from the concrete. By means of these projecting bolts, timbers may be fastened wherever desired and the shaft-hangers lag-screwed in place.

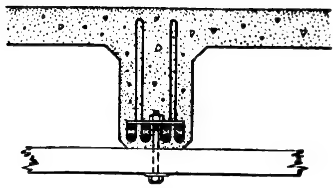


FIG. 101.

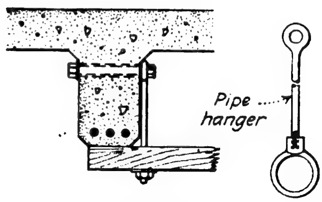


FIG. 102.

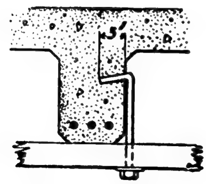


FIG. 103.

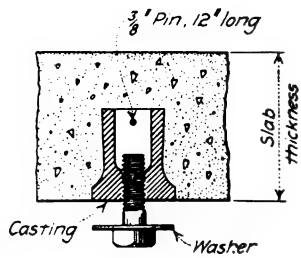


FIG. 104.

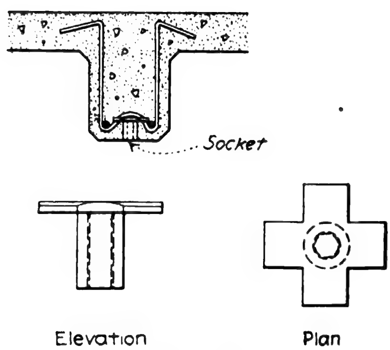


FIG. 105.

The heads of bolts projecting into concrete should be enlarged or bent so that the bolt will not tear out. If desired, a washer or plate may be employed for this purpose underneath the head of the bolt. Figs. 94 and 95 illustrate the use of large washer nuts. These are temporarily held in position by a thin iron tube

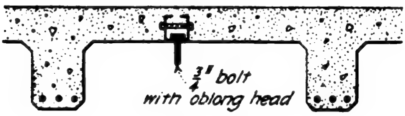


Fig. 106.

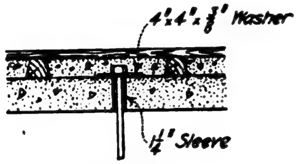


Fig. 107.

resting on the centering and by a bolt projecting through the hole in the bottom of the beam box. The washer nut is tightened down to the top of the tube which secures it firmly in an upright position (see Concrete Engineering Co's. detail shown in Fig. 96).

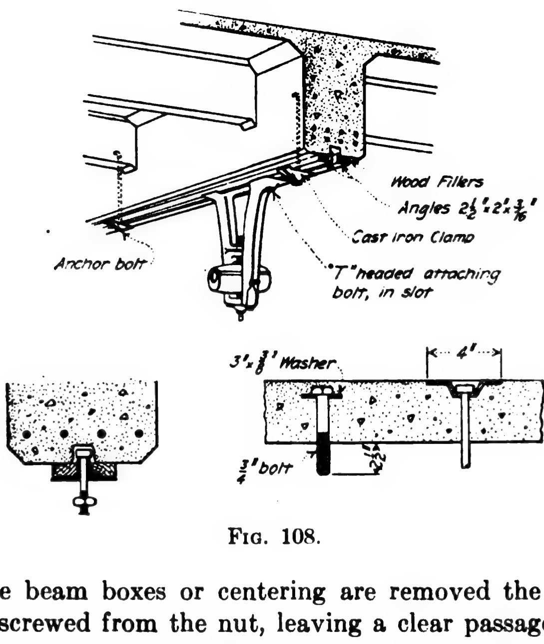


Fig. 108.

When the beam boxes or centering are removed the bolts are easily unscrewed from the nut, leaving a clear passage through the concrete to the nut above. The stirrups shown are to prevent excessive deformation in the beam. By the method

shown in Fig. 95, the shaft-hangers may be securely fastened by bolts.

When projecting bolts are used, it is seldom that all are made use of, and those not used present an unsightly appearance. This is the main objection to the scheme shown in Fig. 97. Saddles and check nuts serve to hold the long bolts in position.

The Unit socket is shown in Fig. 98. This device serves also as a support and spacer for the beam rods, keeping them all

Courtesy of Mr. H. F. Porter, Mfg. Editor of "Factory."

Fig. 109.—Interior Philadelphia Watch Case Co., Riverside, N. J. Inserts for holding sprinkler pipes similar to the insert shown in Fig. 99. Sliding door at the right should be noted.

properly spaced from each other and from the beam box or centering.

A form of adjustable socket for use in beams, principally, is shown in Fig. 99. These castings are made in convenient lengths and the slot in the bottom makes it possible to place hangers or bolts at any desired location along the length of the insert. The casting can be anchored as securely in the concrete as may be necessary by the use of stirrups passed through the open spaces.

Figs. 100 and 101 illustrate two other methods used for attaching shaft-hangers to beams. Fig. 100 is what is known as a pipe slot hanger. Fig. 101 can be made a strong and serviceable support for motors and machinery.

The schemes shown in Figs. 102 and 103 have been used quite extensively by the Turner Construction Co. of New York.

Fig. 104 shows a hanger socket mainly for use in slabs. The casting varies in length with the depth of the slab and is made smaller at the tapped end than at the top, so there will be no

Courtesy of Turner Construction Co.

FIG. 110.—Press room, Robert Gair Factory, Brooklyn, N. Y. Note heavy machinery loads supported on floors designed for 200 lb. per square foot. Also, note sprinkler and wiring connections.

possibility of the binding of the tap-screw when screwed in. The cross-pin shown passes transversely through a cored hole in the upper end of the casting. Fig. 105 shows a somewhat similar type of socket. A bolt through a hole in the form secures it in position during concreting. Figs. 106 and 107 show two other methods used for slabs between beams.

A flexible method of attachment is shown in Fig. 108—a method used in the shops of the United Shoe Machinery Co., Beverly, Mass. The anchor bolts were spaced 3 ft. on centers in all transverse floor and roof girders. A transverse line of bolts alternately spaced 1 ft. and 6 ft. centers was also built in

the middle of each floor and roof panel. In the girder arrangement, the nuts on the lower ends of the anchor bolts engage cast-iron saddles which clamp against pairs of angles with wood fillers. Fig. 108 shows how the "T"-headed attaching bolts

Courtesy of Turner Construction Co.

FIG. 111.—Paper-cutting machines in Gair Building. Note method of attaching shafting.

may move freely along the slot formed by the angles. In the floor panels, the bolts have simple heads or nuts upon the upper ends which bear upon flat washers. In thin slabs the heads bear upon washer plates in the upper surface of the slab (Fig. 108).

It is not good practice to depend upon reinforcing rods for direct support of shaft-hangers, as thereby too great a strain is placed locally on the rods and the concrete under them will become loosened from the continuous pull and vibration of the shafting.

Figs. 109 to 112, inclusive, will give the student a better idea of the manner of attaching shaft-hangers and sprinkler pipes.

Courtesy of Universal Portland Cement Co.

FIG. 112.—Factory building of the Minnesota State Prison, Stillwater, Minnesota. Mushroom system. Note the attachment of sprinkler pipes and shafting.

26. Bedding Machinery.—Machines can readily be connected to the concrete by drilling small holes in the floor at the proper points and setting lag-screws or through-bolts. If through-bolts are required, the holes should be located before the floor is concreted. A permanent level base may be obtained by leveling the machine an inch or two above the foundation proper and grouting it in place. A dam of sand is first built around the machine. Then the grout, in proportions one part cement to one or two parts of sand mixed to the consistency of thick cream, is poured so as to run under the casting. As the mortar hardens it is continually rammed with a rod to prevent shrinkage.

In most cases machinery runs better and lasts longer if placed

on solid foundations, but there are exceptional cases where machines seem to require more or less *give* on their bases, as in certain textile mills. In such cases a wood-finished floor can be laid or wooden timbers fastened to the concrete floor. Sheet cork and linoleum carpet have also been used for this purpose.

27. Waterproof Floors for Fire Protection.—In case of fire, a watertight floor in factory buildings prevents damage from water to the machinery or materials in the stories below. According to *Insurance Engineering*, the damage from water after a fire in fireproof factory buildings is much greater than that from the fire itself. A concrete floor with granolithic surface is practically impervious to water but unless a floor is made self-draining, water will get down through the floor openings. Raised sills should be provided around all openings, whether or not a cement finish is used, and scuppers of ample size should be placed in the outside walls to carry water from the floors to the outside of the building. Fig. 113 is a detail of wall scuppers used in the Liberty Silk warehouses, New York City. The waterproofing of floors may be accomplished by using an impervious finished flooring or by using an under coating of asphalt felt laid in hot asphalt.

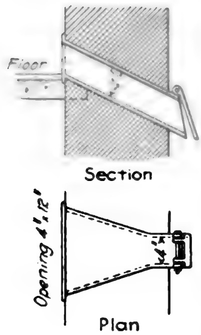


FIG. 113.

28. Tests.—Within the last few years full-sized floor panels of both the monolithic beam and girder and the flat-slab types, have been tested (Fig. 114) for the elastic deformation of the concrete and the steel under increasing loads. The tests consisted in loading, with an increasing load, a number of consecutive panels of a reinforced-concrete building, and of observing the exact contraction or expansion of certain portions of the steel and of the concrete by means of delicate instruments. In order to make such observations, the concrete was removed from the steel in given places, and drill holes were made in the steel thus uncovered. Into these holes the extensometer points were placed. Small holes were also drilled in the concrete for the same purpose. From the elastic deformations obtained as above described and from values obtained for moduli of elasticity of the steel and concrete, the stresses in the members were computed and the results compared with those for which the building was designed.

Only experienced observers were employed in making the tests above described on account of the small deformation in the material. This small deformation makes great care necessary in the manipulation of the instruments if reasonable accuracy is to be obtained. The possibilities of this method have not yet been realized, but it is expected that in the near future important results will come from an analysis of the stresses obtained from a number of these tests.

Courtesy of Ferro-concrete Construction Co.

FIG. 114.—Test of floor in the Wenalden warehouse, Chicago, Ill.

The presence of arch action in flat-slab floors is very clearly shown in Figs. 115 and 116. The diagrams were made from measurements taken in the test of the Chicago warehouse of Larkin Co. (Fig. 71) by Mr. Arthur R. Lord, structural and testing engineer of Chicago. The building was designed in accordance with Chicago's building ordinance which governs flat-slab construction. Fig. 117 shows the location of the gage points on the upper surface of the slab. The diagrams show the upper and lower surfaces of the slab to be in compression for a considerable distance on either side of the point of inflection. The average compression on the section at the point of inflection is indicated as from 150 to 250 lb. per square inch. As in other tests for actual stresses it is evident from the measured stresses that a great deal

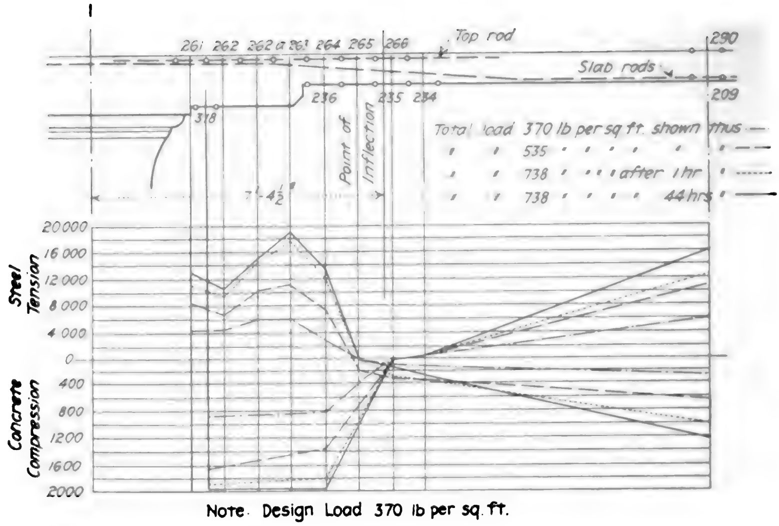


FIG. 115.—Point of inflection readings in diagonal direction across wall panel.

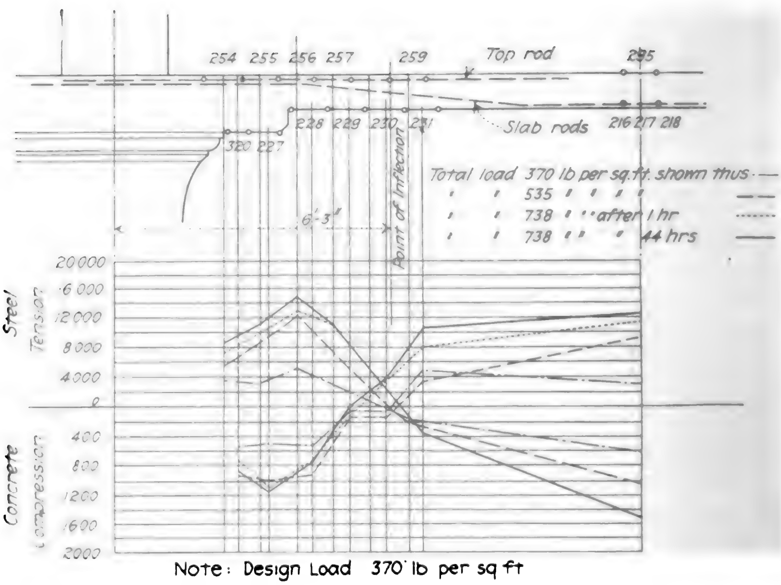


FIG. 116.—Point of inflection readings in long cross direction of wall panel.

of the load is carried by the moment of the plain concrete slab and by arch action. The compressive stresses are altogether greater than the tensile stresses in the steel would lead us to expect.

In the Larkin test, deflections were measured at 24 points. In Fig. 118 three important readings are plotted for the entire period of the test and for one week subsequent to the removal of

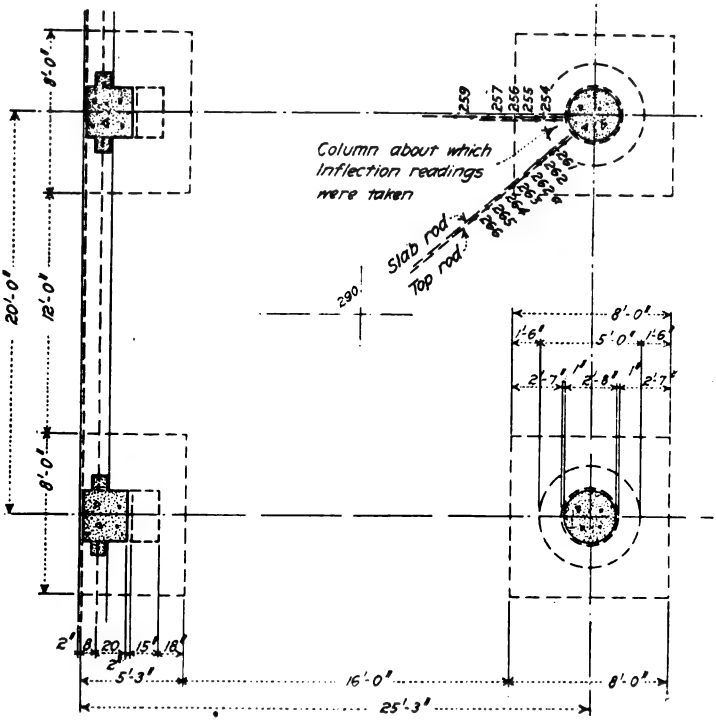


FIG. 117.

the load. These readings were taken at the centers of panels and are very instructive. Mr. Lord in a paper given before the Ninth Annual Convention of the National Association of Cement Users, states:

"Four sets of readings were taken under full load (twice the dead and live design load) and three sets after the load was removed. In Fig. 118 note that the deflection decreased during the last five hours that the full load remained on the floor. A few days after the test load was removed it became necessary to place the cinder fill and floor strips for

the maple flooring and these were held in place by posts and wedges driven hard against the slab above. A material increase in the deflection resulted. At the time the last reading was taken, the posts had been removed a short time, and the weight of the cinder fill and sleepers alone was on the floor."

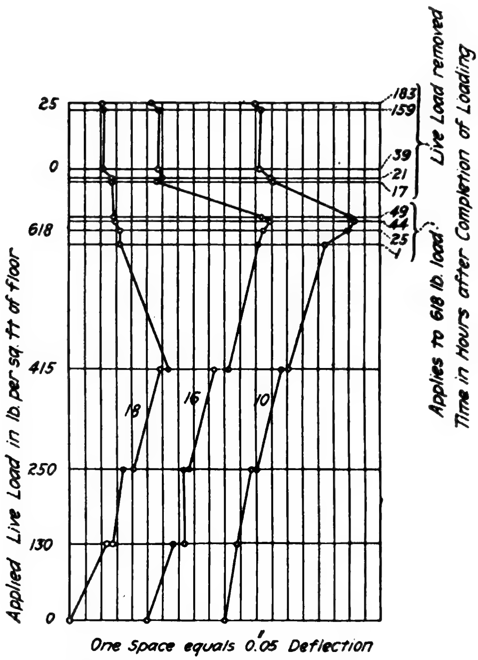


FIG. 118.—Deflection diagram.

In connection with the introduction of the Ransome Unit System (see Art. 19), a series of very interesting tests were made in the Harvard University Laboratory on T-beams of both the monolithic and unit types, reinforced with straight rods only, and with both straight and bent rods. The top of the stem of some of the unit beams were left fairly smooth, as would occur in everyday practice, and, in the other unit beams, deep corrugations were made. All the beams had U-stirrups which extended well up into the slab. In the unit beams, the slab was cast from four to nine days later than the stem. Twenty-eight beams were prepared, but only eleven have so far been tested—the balance being held for a longer-time test.

Professor L. J. Johnson, who made the experiments, expects to publish in due season a complete report of both the short- and long-time tests. Some of the conclusions arrived at by Ransome and Saurbrey (eye-witnesses of these tests), as given in their book "Reinforced Concrete Buildings," are as follows:

"The unit beams stood up as well under the load as the monolithic beams, so that the joint between slab and stem was perfectly adequate, whether corrugated or plain. The general behavior of all these beams up to the point of failure was so much the same that no one, from observation of the beams in the machine, could have pointed out which beams were unit and which monolithic. In fact, they all failed in the customary manner, exhibiting the usual inclined and vertical cracks, and no sliding was noticeable between slab and stem, although carefully looked for.

"The feasibility of the unit beam was established beyond doubt, contrary to what many engineers would probably have expected. In fact, many building regulations throughout the country specify positively that the beam and its superimposed slab must be concreted in one continuous operation. Where improperly designed (or otherwise inadequate), U-bars are used, this rule is undoubtedly highly beneficial, but where proper U-bars are used, the rule is wholly unnecessary. The progress report of the special committee of the American Society of Civil Engineers recommends that the slab be considered effective in compression when "proper bond" is provided between slab and stem; it will be appreciated that this is a much more consistent requirement, although somewhat indefinite. The beams tested so far have shown that the bond provided was adequate, whether the more elaborate method of corrugating the top of the stem was used, or whether the top of the stem was simply left as it was upon completion."

During the last few years a number of experiments have been made to determine the possibility of damage to reinforced concrete structures by stray currents from electric railways and other power sources. A summing up of the general results of these experiments is given in the Joint Committee's report at the end of this volume.

29. **Basement Floors.**—A basement floor in dry ground is usually made of 1:3:5 concrete, 3 or 4 in. thick. Usually no wearing surface is needed other than the ordinary concrete troweled to a hard finish, but, where considerable wear is expected, the usual mortar-coat may be laid as on the upper floors. To prevent shrinkage cracks, the floor should be divided into blocks about 8 or 10 ft. square. This may be accomplished by laying

alternate blocks, and then filling in the intermediate ones after the adjoining concrete has set.

It is not safe to depend upon the concrete itself being water-tight. If the basement is below tide water or ground water level, a layer of waterproofing consisting of three to six layers of waterproof felt (cemented together and to the concrete by coal-tar pitch or asphalt) should be spread on the concrete and carried up in continuous sheets on the walls to above water level. The whole surface should then be covered with another layer of concrete at least 3 or 4 in. thick.

The earth under the basement floor should be well drained, and drains of tile pipe, or of screened gravel and stone, may be placed in trenches just below the floor. Sometimes it is necessary to cover the entire area with cinders or stone; and sometimes the concrete must be made extra thick, or reinforcement added, to resist the upward pressure of the water.

CHAPTER V

TYPES OF REINFORCEMENT

This chapter is inserted with the intention of giving the student an idea of the different types of reinforcement that he will find in practice. No claim is made that all the types or systems are included. It is thought, however, that those shown are representative and that the student will be able to discern for himself any advantages or disadvantages which may be found in the different types.

30. Wire Fabric.—This material is used to a considerable extent in floor and roof slabs and has been found to possess many valuable qualities. Wire fabric is made of steel wires crossing generally at right angles and secured at the intersections. The heavier wires run lengthwise and are called carrying wires; the lighter ones cross these and are called distributing or tie wires. One distinct advantage in the use of fabric is that it preserves uniform spacing of the steel.

Wire fabric is also used to some extent as reinforcement for walls, partitions, and conduits.

The steel-wire gage used by practically all the steel-wire manufacturers of the United States under various names is given in the following table:

STEEL WIRE GAGE

Diameter inches	Steel-wire gage	Diameter inches	Area square inches	Pounds per foot	Feet per pound
$\frac{1}{2}$	....	0.500	0.19635	0.6625	1.50
	$\frac{7}{0}$	0.490	0.18857	0.6363	1.51
$\frac{15}{32}$	....	0.468	0.17202	0.5804	1.72
	$\frac{6}{0}$	0.460	0.16619	0.5608	1.78
$\frac{7}{16}$	....	0.437	0.14998	0.5061	1.97
	$\frac{5}{0}$	0.430	0.14532	0.4901	2.04
$\frac{13}{32}$	....	0.406	0.12946	0.4368	2.28
	$\frac{4}{0}$	0.393	0.12130	0.4094	2.44
$\frac{3}{8}$	....	0.375	0.11044	0.3726	2.68
	$\frac{3}{0}$	0.362	0.10292	0.3473	2.87

STEEL WIRE GAGE—*Continued*

Diameter inches	Steel-wire gage	Diameter inches	Area square inches	Pounds per foot	Feet per pound
$\frac{11}{32}$	....	0.343	0.09240	0.3117	3.20
	2/0	0.331	0.08604	0.2904	3.44
$\frac{5}{16}$	....	0.312	0.07645	0.2579	3.87
	0	0.307	0.07402	0.2497	4.00
	1	0.283	0.06290	0.2123	4.71
$\frac{9}{32}$	....	0.281	0.06210	0.2092	4.78
	2	0.263	0.05432	0.1834	5.45
$\frac{1}{4}$	....	0.250	0.04908	0.1656	6.03
	3	0.244	0.04675	0.1578	6.33
	4	0.225	0.03976	0.1342	7.45
$\frac{7}{32}$	....	0.218	0.03732	0.1259	7.94
	5	0.207	0.03365	0.1135	8.81
	6	0.192	0.02895	0.0977	10.23
$\frac{3}{16}$	....	0.187	0.02746	0.0926	10.79
	7	0.177	0.02460	0.0830	12.04
	8	0.162	0.02061	0.0696	14.36
$\frac{5}{32}$	....	0.156	0.01911	0.0644	15.52
	9	0.148	0.01720	0.0580	17.24
	10	0.135	0.01431	0.0483	20.70
$\frac{1}{8}$	....	0.125	0.01227	0.0414	24.15
	11	0.120	0.01130	0.0382	26.17
	12	0.105	0.00865	0.0292	34.24
$\frac{3}{32}$	....	0.093	0.00679	0.0229	43.66
	13	0.092	0.00664	0.0224	44.64
	14	0.080	0.00502	0.0169	59.17
	15	0.072	0.00407	0.0137	72.99
	16	0.063	0.00311	0.0105	95.23

The manner of securing the intersections of wire fabric has given rise to a number of different types, several of the principal ones of which are given below. The manufacturers of each of these forms will furnish fabric in special size of wire and mesh if desired. Much of that which follows is taken directly from catalogs.

Welded Wire Fabric.—Welded wire fabric, Fig. 119, manufactured by the Clinton Wire Cloth Co., is a wire mesh made up of a series of parallel longitudinal wires, spaced a certain distance apart and held at intervals by means of transverse wires, arranged at right angles to the longitudinal ones, and welded to them at the points of intersection by a patented electrical process. Longitudinal wires can be spaced on centers of 2 or more inches, in steps of

½ in. Transverse wires can be spaced on centers of 1 to 18 in. inclusive, in steps of 1 in. The following table shows the sizes and tensile strength of the wire used. Rolls kept in stock vary in length between 150 and 200 ft. and between 62 and 86 in. in width.

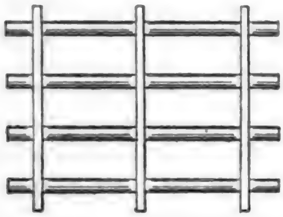


Fig. 119.—Welded wire fabric.

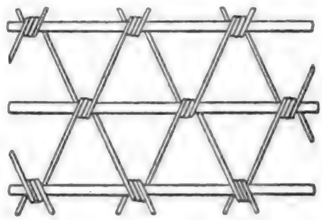


Fig. 120.—Triangle-mesh wire fabric.

WELDED WIRE FABRIC

Size per steel wire gage No.	Diam. of one wire in.	Sectional area of one wire sq. in.	Weight per linear foot of one wire lb.	Tensile strength of one wire lb.	Tensile strength of longitudinal wires only, in 1 ft. of the width of welded fabric, when spaced as below:				
					2-in. centers lb.	3-in. centers lb.	4-in. centers lb.	5-in. centers lb.	6-in. centers lb.
0	0.3065	0.07378	0.2506	4721	28,326	18,884	14,163	....	9442
1	0.2830	0.06290	0.2136	4025	24,150	16,100	12,075	....	8050
2	0.2625	0.05411	0.1838	3463	20,778	13,852	10,389	....	6926
3	0.2437	0.04664	0.1584	2984	17,904	11,936	8,952	7161	5968
4	0.2253	0.03986	0.1354	2551	15,306	10,204	7,653	6122	5102
5	0.2070	0.03365	0.1143	2153	12,918	8,612	6,459	5167	4306
6	0.1920	0.02895	0.0983	1852	11,112	7,408	5,556	4445	3704
7	0.1770	0.02460	0.0835	1574	9,444	6,296	4,722	3777	3148
8	0.1620	0.02061	0.0700	1319	7,914	5,276	3,957	3165	2638
9	0.1483	0.01727	0.0586	1105	6,630	4,420	3,315	2652	2210
10	0.1350	0.01431	0.0486	915	5,490	3,660	2,745	2196	1830
11	0.1205	0.01140	0.0387	729	4,374	2,916	2,187	1749	1458
12	0.1055	0.00874	0.0296	559	3,354	2,236	1,677	1341	1118

Triangle-mesh Wire Fabric.—Triangle-mesh steel-wire fabric, manufactured by the American Steel and Wire Co., is made with both single and stranded longitudinal or tension members. That with the single wire longitudinal is made with one wire varying in size from a No. 12 gage up to and including a ½-in. diameter, and that with the stranded longitudinal is composed of two or three wires varying from No. 12 gage up to and including No. 4 wires

stranded or twisted together with a long lay. These longitudinals either solid or stranded are invariably spaced 4 in. centers, the sizes being varied in order to obtain the desired cross sectional area of steel per foot of width (see Fig. 120).

The transverse or diagonal cross wires are so woven between the longitudinals that triangles are formed by their arrangement. These diagonal cross or transverse wires are woven either 2 or 4 in. apart, as is desired. Triangle-mesh wire reinforcement is made in lengths of 150 to 600 ft. and in widths from 18 to 58 in. The table following shows the number and gage of wires and the areas per foot width when the longitudinals and cross-wires are spaced 4 in. on centers.

TRIANGLE-MESH WIRE FABRIC

Style number	Gage of longitudinal wires	Gage of cross-wires	Sectional area in sq. in. of longitudinal wires per foot width	Sectional area in sq. in. of cross-wires per ft. width	Cross-sectional area in sq. in. per foot width
4	6	14	0.087	0.025	0.102
5	8	14	0.062	0.025	0.077
6	10	14	0.043	0.025	0.058
7	12	14	0.026	0.025	0.041
23	¼-in.	12½	0.147	0.038	0.170
24	4	12½	0.119	0.038	0.142
25	5	12½	0.101	0.038	0.124
26	6	12½	0.087	0.038	0.110
27	8	12½	0.062	0.038	0.085
28	10	12½	0.043	0.038	0.066
29	12	12½	0.026	0.038	0.049
31	4	12½	0.238	0.038	0.261
32	5	12½	0.202	0.038	0.225
33	6	12½	0.174	0.038	0.196
34	8	12½	0.124	0.038	0.146
35	10	12½	0.086	0.038	0.109
36	12	12½	0.052	0.038	0.075
38	4	12½	0.358	0.038	0.380
39	5	12½	0.303	0.038	0.325
40	6	12½	0.260	0.038	0.283
41	8	12½	0.185	0.038	0.208
42	10	12½	0.129	0.038	0.151
43	12	12½	0.078	0.038	0.101

Tie-locked Wire Fabric.—Tie-locked fabric, Fig. 121, is manufactured by the Cargill Manufacturing Co., in stock lengths from 50 to 200 ft. and stock widths of 4, 5, and 6 ft. This fabric is used in combination with steel cable distributing members.

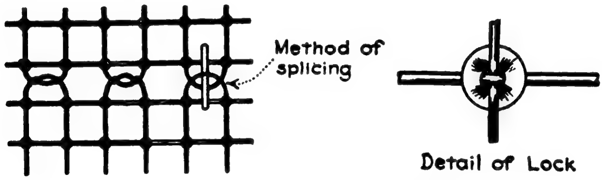


FIG. 121.—Tie-locked fabric.

In the construction after the centering or falsework is in place, the cables are anchored to a wall or beam (see Fig. 122) and extend to the opposite end of the building. They are spaced according to the size of spans and the desired carrying capacity of the slabs. After drawing the cables reasonably tight, they are then secured to other anchors provided for that purpose and the wire

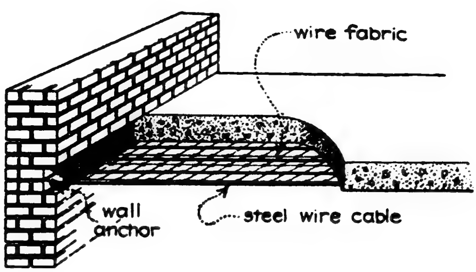


FIG. 122.—The International system.

fabric is placed directly upon the cables. The anchors are provided for the sole purpose of holding the materials to their exact position while the concrete is being placed, and upon the completion of the construction the anchors have no further duty to perform.

The following table shows the stock styles of tie-locked wire fabric. The standard cables used are composed of 7 strands No. 9 gage steel wire and 4 strands No. 7 gage.

TIE-LOCKED WIRE FABRIC

Style	Gage of longitudinal wire	Gage of cross-wire	Mesh in inches
A-1	9	11	6×6
A-2	9	11	5×6
A-3	9	11	4×6
B-1	7	9	6×6
B-2	7	9	5×6
B-3	7	9	4×6
C-1	9	11	6×12
C-2	9	11	5×12
C-3	9	11	4×12

Lock-woven Wire Fabric.—Lock-woven wire fabric, Fig. 123, is made by W. N. Wight & Co. It is similar to the welded wire fabric, except that instead of electric welding the intersections are bound together by winding them with soft wire.

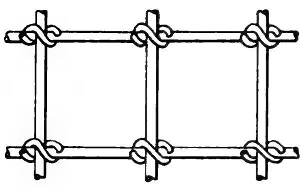


FIG. 123.—Lock-woven wire fabric.

This fabric is made from 56 to 88 in. wide and is put up in rolls of from 330 to 500 lin. ft. The wire used is of high carbon steel. The longitudinal wires are usually spaced 4 in. on centers and the transverse wires 5 in. The table which follows gives the different styles in stock.

LOCK-WOVEN WIRE FABRIC

Style	Gage of longitudinal wires.	Gage of cross-wires	Mesh in inches
A	10	9	4×6
B	8	10	4×6
C	6	10	4×6
D	4	10	4×6
E	3	10	4×6

Staple-locked Wire Fabric.—A rectangular-mesh staple-locked fabric (Fig. 124) is manufactured by the American System of Reinforcing. The wire used is of high carbon steel and is secured

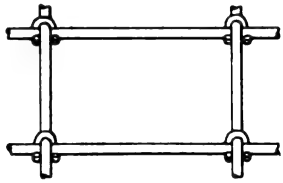


Fig. 124.—Staple-locked wire fabric.

at the intersections by No. 14 wire. Standard sizes are shown in the following table. The fabric comes in standard widths of 3, 4, and 5 ft., 200 lin. ft. in a roll.

STAPLE-LOCKED WIRE FABRIC

Gage of longitudinal wires	Gage of cross-wires	Mesh in inches
9	11	4×12
9	11	4×6
9	11	6×6
7	11	4×12
7	11	4×6
7	11	6×6

31. Expanded Metal.—Expanded metal invented by Mr. John. T. Golding is one of the oldest forms of sheet reinforcement (Fig. 125). Sheet steel is slit in a special machine and then pulled

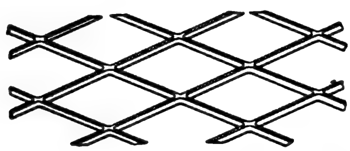


Fig. 125.—Expanded metal.

out or expanded so as to form a diamond mesh. The standard sizes and gages as adopted by the Associated Expanded Metal Companies are shown in the following table:

EXPANDED METAL

Designation: Mesh inches	Designation: Gage No.	Designation: Strand standard or extra	Section in sq. in. per foot width	Weight per sq. ft. in pounds	Size of standard sheets, feet	No. of sheets in a bundle	No. of sq. ft. in bundle of 8 ft. length
½	18	Standard...	0.209	0.74	4 or 5× 8	5	....
¾	13	Standard ...	0.225	0.80	6 ×8 or 12	5	240
1½	12	Standard ...	0.207	0.70	4 ×8 or 12	5	160
2	12	Standard ...	0.166	0.56	5 ×8 or 12	5	200
3	16	Standard	0.083	0.28	6 ×8 or 12	10	480
3	10	Light.......	0.148	0.50	6 ×8 or 12	5	240
3	10	Standard ...	0.178	0.60	6 ×8 or 12	5	240
3	10	Heavy......	0.267	0.90	4 ×8 or 12	5	160
3	10	Ex. heavy ..	0.356	1.20	6 ×8 or 12	3	144
3	6	Standard ...	0.400	1.38	5 ×8 or 12	3	120
3	6	Heavy......	0.600	2.07	5 ×8 or 12	3	120
4	16	Old style ...	0.093	0.42	4½×8 or 12	6	216
6	4	Standard ...	0.245	0.84	5 ×8 or 12	5	200
6	4	Heavy......	0.368	1.26	5 ×8 or 12	3	120

32. Rib Metal.—Rib metal is manufactured by the Trussed Concrete Steel Co., and consists of nine longitudinal ribs rigidly connected by light cross members. It is made from a sheet of

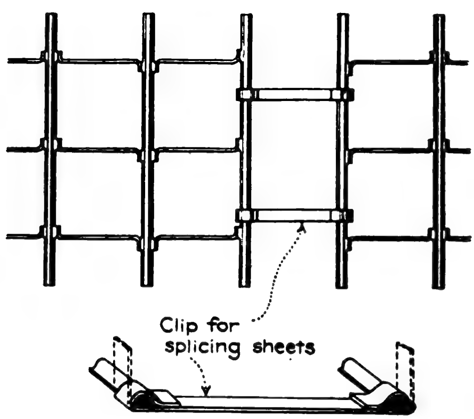


FIG. 126.—Rib metal.

metal, flat on one side and corrugated on the other. Strips of the metal adjacent to the ribs are stamped out, and the sheet is drawn out into square meshes (Fig. 126). The standard sheets are manufactured with meshes of from 2 to 8 in. and in lengths

12, 14, and 16 ft. The properties of rib metal are given in the table which follows.

RIB METAL

Size No.	Width of standard sheet inches	Sq. ft. per linear foot of standard sheet	Area per ft. width sq. in.	Ult. tensile strength per foot of width	Safe tensile strength per foot of width pounds
2	16	1.33	0.54	38,880	9,720
3	24	2.00	0.36	25,920	6,480
4	32	2.67	0.27	19,440	4,860
5	40	3.33	0.216	15,552	3,888
6	48	4.00	0.18	12,960	3,240
7	56	4.67	0.154	11,088	2,772
8	64	5.33	0.135	9,720	2,430

Area of one rib = 0.09 sq. in.
Ultimate tensile strength = 6480 lb.
Safe tensile strength = 1620 lb.

33. Reinforcing Systems.—Following are a number of reinforcing systems in common use:

Hennebique System.—One of the pioneers in concrete construction in Europe is Mr. Hennebique, in France, and the system which still bears his name is shown in Fig. 127.

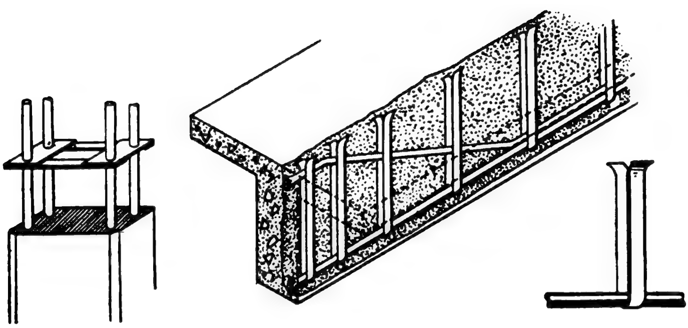


FIG. 127.—Hennebique system.

Columbian System.—The Columbian system as applied to the monolithic beam-and-girder type of construction is shown in Fig. 128.

Kahn Trussed-bar System.—The Kahn Trussed Bar, Fig. 129, named for its inventor, is rolled with flanges, which are bent up to resist the shear in the beam. For continuous beams, inverted

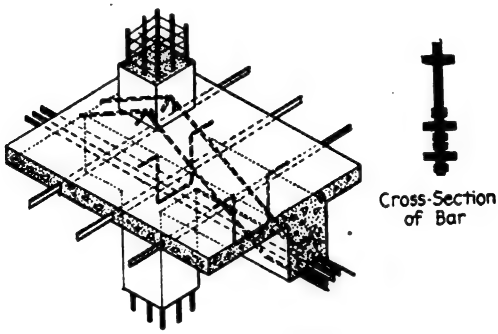


FIG. 128.—Columbian system.

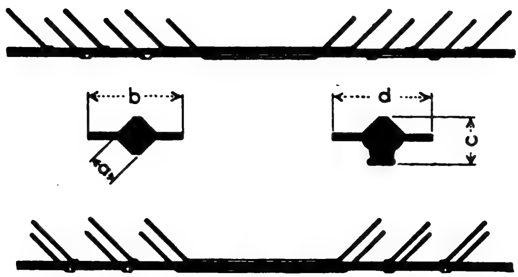


FIG. 129.—Kahn trussed bars.

bars are placed over the supports in the upper part of the beam, extending over the region of tension. Properties of Kahn trussed bars are shown in the following table:

KAHN TRUSSED BARS

Size in inches $a \times b$	Weight in pounds per foot	Area	Length of diagonals in inches	
			Standard	Special
		SQUARE SECTION BARS		
$\frac{1}{2} \times 1\frac{1}{2}$	1.4	0.41	6	8, 12
$\frac{3}{4} \times 2\frac{3}{16}$	2.7	0.79	12	8, 18, 24, 30

NEW SECTION BARS

1⅛×2⅛	4.8	1.41	24, 30	12, 18, 36
1⅜×2⅜	6.8	2.00	30	18, 24, 36
2 ×3⅛	10.2	3.00	30	24, 36, 48

Note.—8, 12, 18, 24, 30, 36, and 48-in. diagonals are sheared alternately. 6-in. diagonals only are sheared opposite.

Cummings System.—The Cummings System is shown in Fig. 130. U-shaped stirrups are used on the girder frame shown. They are shipped flat with the longitudinal reinforcement, but are bent up to an inclined position on the work. The rods are

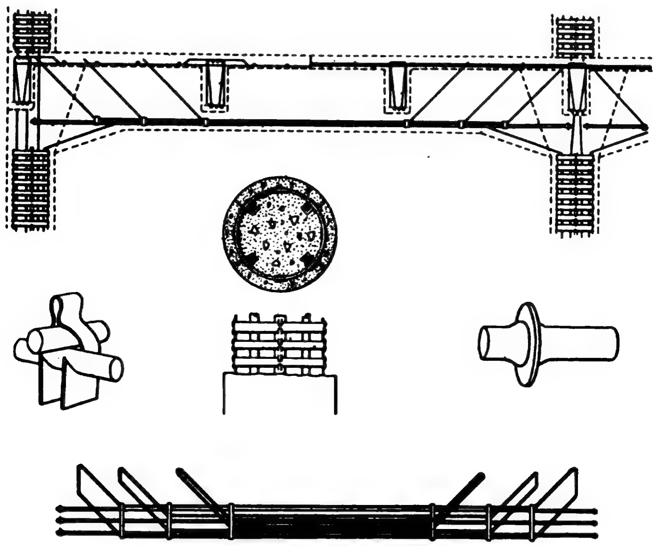


FIG. 130.—Cummings system.

held together by means of a patented chair. In the Cummings hooped column, each hoop is securely attached to the upright rods. The hoops are made of flat steel, bent to a circle, with the ends riveted or welded together in such a manner that the ends of the hoops protrude at right angles to keep them the proper distance from the mold.

Unit System.—Fig. 131 shows the Unit System of Reinforcing. The girder frames are not stock frames but are built to meet the engineer's or architect's plans. Unit girder frames are provided

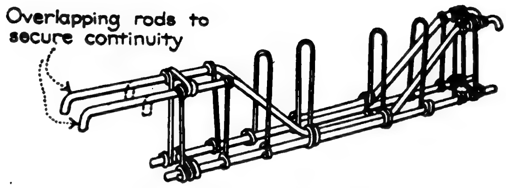


Fig. 131.—Unit system.

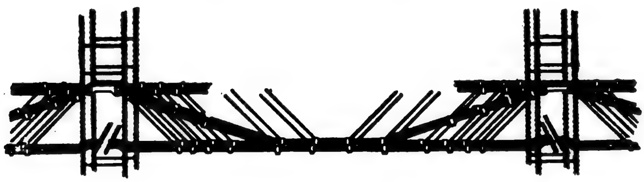


Fig. 132.—Pin-connected system.

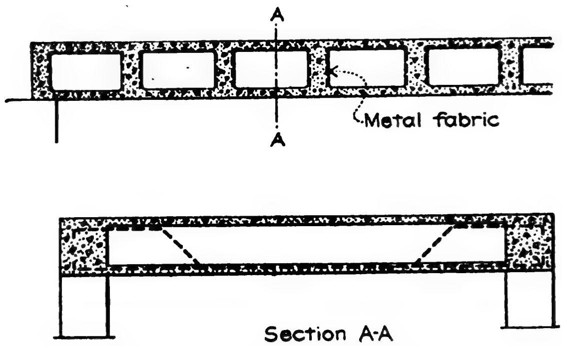


Fig. 133.—Merrick floor system.

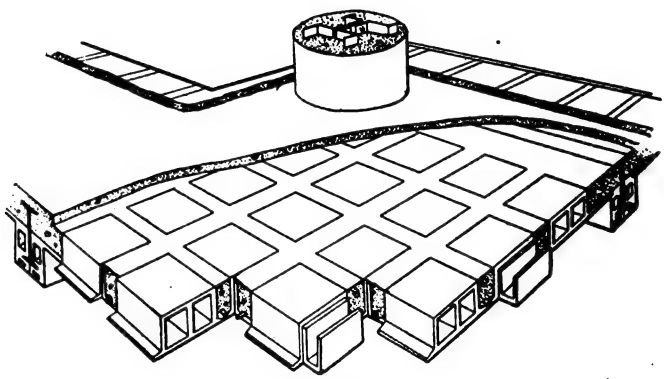


Fig. 134.

with overlapping rods for continuous beams to reinforce against negative moment.

Mushroom System.—See Figs. 67 and 68.

Pin-connected System.—Reinforcement in the Pin-connected System consists of bars made into a truss and ready for placing in the forms. (See Fig. 132.)

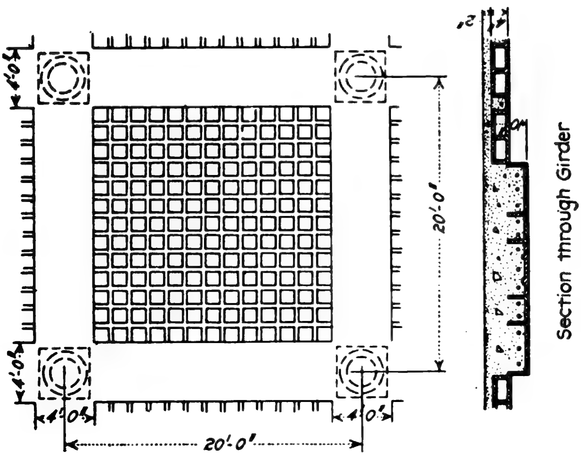


FIG. 135.—Corr-tile floor applied to wide girder system of reinforced-concrete framing.

Merrick Floor System.—The Merrick System is designed to lighten the weight of the concrete slab by using a hollow floor construction, as shown in Fig. 133. Directly upon the forms a 2-in. layer of concrete is placed, and before this has set, oblong

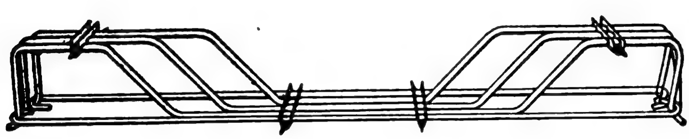


FIG. 136.—Luten truss.

boxes of metal fabric of small mesh are laid horizontally, with the reinforcing rods in the spaces between them. The concrete is filled in between the boxes and around the reinforcing rods, and then poured over the top to form the floor.

Corr-tile System.—Corr-tile Two-way Floor Construction is shown in Figs. 134 and 135. Fig. 134 shows the Corr-tile used in

connection with steel frame construction, and Fig. 135 shows it applied to a wide girder system of reinforced-concrete framing. In Corr-tile construction the standard arrangement of tile known

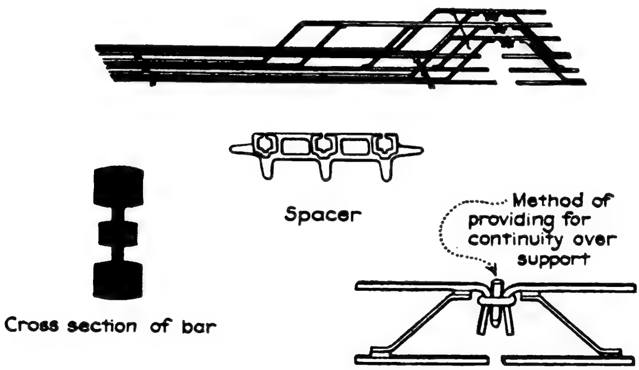


Fig. 137.—Xpantruss system.

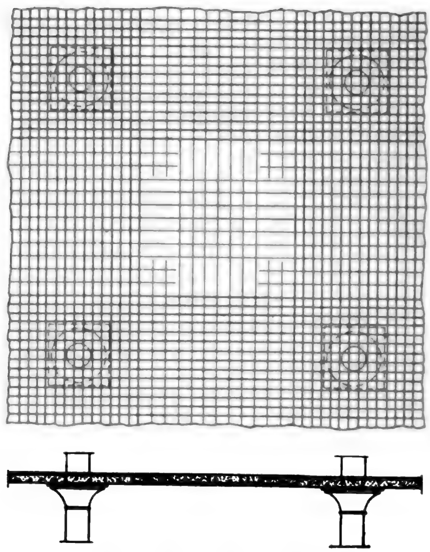
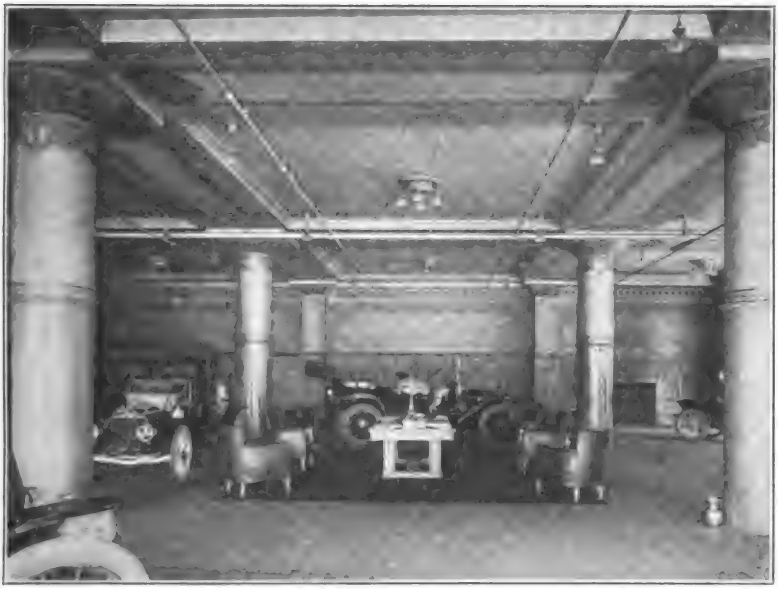


Fig. 138.—Akme system.

as System A, consists of flanged tile, block tile, and channel tile, in alternate rows. Small squares of tile are employed to fill the voids at the intersections of ribs and provide an unbroken tile ceiling surface. Except for the flanges, the block tile are similar

in shape and section to the standard floor and partition tile in common use. The flanges occurring on two sides, as indicated in Fig. 134 are 1½ in. in width, and serve to properly space the tile in one direction and to effectively fire-proof the concrete ribs formed above them. Flanged block tile are manufactured in 4, 6, 8, 9, 10, and 12-in. depths, and are sufficiently scored on all exterior surfaces to insure thorough bond with concrete or plaster.



Courtesy of Universal Portland Cement Co.

Fig. 139.—Reinforced-concrete Automobile Bldg., Studebaker Automobile Co., Chicago, Ill. Akme system of floor construction.

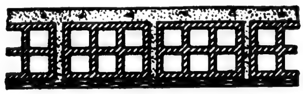
Tile of 5-in. and 7-in. depth can be obtained on special demand. Channel tile are obtainable in various depths, corresponding to the several depths of block tile. A uniform width of 3 in. between sides is observed in all sizes of channels.

Luten Truss.—The Luten truss is shown in Fig. 136. The bars are rigidly locked together to form the truss by a clamp, with a wedge that is self-locking when driven home. The truss is especially adapted to highway culverts and bridges and is put out by the National Concrete Company.

Xpantrus System.—The truss by this name is shown in Fig. 137 and is applicable chiefly to beams, girders, and heavy slabs.

This system is patented by The Consolidated Expanded Metal Companies.

Akme System.—This system (Figs. 138 and 139) is a form of girderless floor construction. In this system of construction all the stresses, due to the weight of the structuie and the loads to be carried, are determined by the usual methods of calculation, as



End View

FIG. 140.—Johnson system.

the reinforcement is placed in two directions only. The Akme System is controlled by the Condron Company of Chicago.

Johnson System.—The Johnson System Floor is constructed chiefly of terra-cotta hollow-tile blocks, as shown in Fig. 140. The steel reinforcement is in the form of steel wire mesh, embedded in a course of concrete under the tile.

PROBLEM

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8. Design an interior floor bay to support a live load of 200 lb. per square foot with the columns spaced 16 ft. by 16 ft. on centers and with one intermediate beam equally spaced. Make the same assumptions and use the same working stresses as are employed in the two-intermediate-beam design of Art. 17.

CHAPTER VI

ROOFS

Reinforced concrete roofs are designed in the same manner as floors and the complete design of a roof bay in Art. 38 will give all that is necessary on the subject of general design. Fig. 141 is a construction view of a one-way hollow-tile roof.

34. Concrete Roof Surfaces.—Although concrete roofs have been designed to be impervious without a covering of any other

Courtesy of Ferro-concrete Construction Co.

FIG. 141.—Construction view of hollow-tile roof, Heekin Building, Cincinnati, Ohio.

material, it seems to be the general opinion that it is difficult to secure absolute imperviousness in this type of roof because of the likelihood of shrinkage cracks. Some engineers, however, believe that the cracks may be kept so minute, by properly reinforcing against all the tensile stresses due to expansion and contraction, that the roof will stay waterproof. In fact, during the past few

years some large buildings have been erected on this theory, and so far have shown no signs of leakage; but it is conservative to say that a longer test is needed to give conclusive results. There is no doubt, however, that a concrete roof without a special covering of any sort is at least well adapted to all structures where absolute imperviousness is not essential, as in train sheds, reservoir roofs, and in *out of door* structures in general.

Shrinkage joints are sometimes introduced in concrete roof slabs. These joints are made water-tight by the introduction of a trough-shaped strip of copper or lead which will take contraction and expansion. If a sufficient number of joints of this nature are made, shrinkage cracks will undoubtedly be reduced to a minimum. To make shrinkage joints the slab must be poured separately from the beams and girders, and no bond should be allowed between the slab and the roof framing. If steel beams are used, there is no danger, but in all-concrete construction, galvanized strips, well oiled, or some equivalent, should be introduced.

A maximum amount of reinforcement against shrinkage cracks will be of little avail in a concrete roof unless the concrete itself is impervious. Methods of waterproofing concrete may be roughly classified as follows: (1) Use of a rich-concrete mixture; (2) use of waterproof coatings or washes; (3) admixture of substances designed to produce impermeability. It is often advisable to combine two or more of these methods.

A roof slab without protective covering should preferably be of a rich mix of concrete of such proportions of sand and stone as to produce maximum density. The method of obtaining maximum density outlined in Art 8, Volume I, may be employed to secure impermeability, if the work is of considerable magnitude. For maximum water-tightness, however, a mortar or concrete may require a slightly larger proportion of fine grains in the sand than for maximum density or strength, but otherwise the general principles discussed in the article on proportioning by mechanical analysis are applicable. Proportions 1:2:4 will generally give satisfaction if the aggregates are well graded. Gravel is preferable to broken stone in making water-tight concrete

Experiments show that permeability of concrete decreases with time and that ordinary wet mixtures are much less permeable than dry mixtures. It has also been found that the light joggling which is necessary to compact a wet concrete, greatly increases its impermeability. Cracks must be prevented, and if the entire

roof slab cannot be laid in one continuous operation, an effective bond should be made between successive days' work.

Plastering a concrete roof surface with a layer of rich mortar has been found effective when care has been taken to place the same before the concrete has taken its final set. If such care is not exercised, variation in temperature and moisture between the concrete and plaster will be almost certain to cause a separation. Troweling concrete to a dense, hard surface, in the same manner that granolithic work is troweled, makes concrete impervious and nearly equal to a surfacing of rich mortar.

The methods employed to waterproof concrete by using washes or by introducing foreign ingredients into the mixture will be considered in Chapter XXIII. The general remarks made there will apply to roofs as well as to other types of structures.

A hard wearing surface is sometimes required in roof construction—for example, when it forms the floor of a roof garden. The roof surface in such a case should have a granolithic finish the same as in floors, and a form of bitumen emulsion mixed with cement mortar may be used in making the surface impervious.

It is not good practice to use a concrete roof without covering on steep slopes, as the slabs must then either be laid quite dry (which does not favor imperviousness) or else top forms must be used to retain the concrete. The latter method is slow and expensive.

35. Separate Roof Coverings.—The felt- and gravel-roof is the most common and efficient form of roof covering. The covering is composed of layers of waterproof felt, cemented together and to the concrete by coal-tar pitch or asphalt. The method followed is to lay from four to eight thicknesses of felt over the roof surface, each ply being cemented to the preceding layer, and the entire surface then floated with a heavy, flowing coat of pitch or asphalt, in which (whileh ot) clean, dry, uniformly screened gravel or slag is embedded, sufficient in quantity to cover the surface thoroughly.

Asphalt has in the past been preferred to coal-tar pitch as a binding medium, but at the present time coal-tar products appear to be satisfactory when made to contain a large percentage of carbon, and are being used by many in preference to asphalt. Asphalt, however, seems to be much superior to coal-tar pitch in ready roofings.

The following specification for a five-ply tar and gravel or slag

roof has been taken from the Proceedings of the American Railway Engineering and Maintenance of Way Association, 1910, and is essentially the Barrett Company's specification.

"There shall be used five (5) thicknesses of saturated felt weighing not less than fourteen (14) lb. per one hundred (100) sq. ft., single thickness; not less than two hundred (200) lb. of pitch; and not less than four hundred (400) lb. of gravel or three hundred (300) lb. of slag from ¼- to ⅝-in. size, free from dirt, per one hundred (100) sq. ft. of completed roof.

"The material shall be applied as follows: First, coat the concrete with hot pitch mopped on uniformly. Second, lay two (2) full thicknesses of tarred felt, lapping each sheet seventeen (17) in. over the preceding one, and mop with hot pitch the full width of the seventeen (17)-in. lap, so that in no case shall felt touch felt. Third, coat the entire surface with hot pitch, mopped on uniformly. Fourth, lay three (3) full thicknesses of felt, lapping each sheet twenty-two (22) in. over the preceding one, mopping with hot pitch the full width of the twenty-two (22) in. lap between the plies, so that in no case shall felt touch felt. Fifth, spread over the entire surface of the roof a uniform coat of pitch, into which, while hot, imbed the gravel or slag. The gravel or slag in all cases must be dry."

The coal-tar pitch or asphalt is the life of the roof, particularly in the topcoating. The gravel or slag should be applied liberally, in order to completely bury the coat of waterproof material and protect it from injury due to walking on the roof, and from the action of the sun. In the following statements asphalt only will be mentioned, but it must be understood that the same statements will apply to coal-tar pitch.

In waterproofing, the object of using saturated felt is merely to provide a medium to hold the asphalt together, and thus allow for expansions, contractions and settlings. The greatest care and judgment must be exercised in laying felt to see that it is properly stretched, laid smoothly, and that no wrinkles appear. All of the roofing felts on the market are made of wool or flax, saturated with a preserving material that will harmonize with the asphalt. Saturated wool roofing felts are compact, and although they absorb little of the asphalt, they hold it in repeated layers to constitute the body material of the roof. The flax felt is porous and of strong texture, and absorbs the asphalt, holding it within its fibers. Unsaturated burlap or canvas can be made to hold asphalt if first run through a bath of liquid asphalt or tar.

The greater the amount of felt and asphalt used, the greater

the life of the roof. However, if too much asphalt is used on top, it will run; hence the flatter the roof, the greater the life. No roof should have less than 100 lb. of asphalt per 100 sq. ft.

What is known as a felt and gravel roof should not be used in cold climates, on a surface of greater pitch than about 3 in. to the foot. In hot climates 1 in. to the foot is about the proper maximum. Where the roof is of greater pitch, the gutters may be put in with galvanized iron, copper, tin or plies of felt and asphalt, and the steeply pitched surface covered with clay tile or slate shingles with lap joints. Vitrified clay tile are made in a great variety of forms, flat, ribbed, and corrugated; but those of some interlocking pattern are best. Nailing strips (1 in.$\times$2 in.) should be inserted in the concrete under each row of tile or slate, but it is important that they should be so placed as not to affect the strength of the roof slab. Slate or tile should not be used on a surface with a pitch of less than 6 in. to the foot.

A most durable roof covering for flat surfaces (slope not exceeding $\frac{1}{4}$ in. to the foot) is vitrified roofing tile embedded in asphalt. The tile consists of flat rectangular terra-cotta tile about 1 in. to $1\frac{1}{4}$ in. thick, bedded in hot asphalt, on top of four to six thicknesses of felt of the kind mentioned above. The joints between the tiles should be filled with asphalt. Slate tiles also make a good wearing surface. They are usually $\frac{7}{8}$ in. to 1 in. thick, by 12$\times$12 in. in area.

Tin, corrugated iron, and copper roofings are sometimes placed on reinforced-concrete buildings. These roofings do no have a long life if laid directly on the concrete, but seem to give satisfaction when a wooden sheathing is used as a separator. Copper is expensive as compared with other types of roofing and is usually employed only on very costly buildings.

There are many roofings on the market which come ready to lay, made on the principle of the built-up roof. These roofings are especially valuable for use in small and isolated buildings, as they do not require the expert help which is necessary for a built-up roof. In a preparation which comes rolled, however, it is impossible to obtain a great deal of waterproof material, and there is an additional weakness in some coverings of this type due to the fact that the waterproofing sheets are nailed together with a short exposed lap. Expansion and contraction of the body material in the course of time opens holes alongside the nails and permits water to enter. In spite of adverse criticism, however,

there are many ready roofings which seem to give satisfaction and are used frequently on pitched surfaces, and to some extent on flat slopes.

A number of types of metal and composition shingles are on the market. Reinforced-concrete slabs from 5 to 6 ft. long have been used for this purpose.

36. Drainage.—Felt and gravel roofs should have a pitch of at least $\frac{3}{8}$ in. to 1 ft. in order to provide proper drainage. Flat tile, however, may be employed on surfaces with a very slight pitch—preferably not over $\frac{1}{4}$ in. per foot.

Gussets in flat roofs are generally formed by placing a well-tamped cinder filling over the concrete slab and then laying a 1- to 2-in. surface of cinder or stone concrete. This type of roof

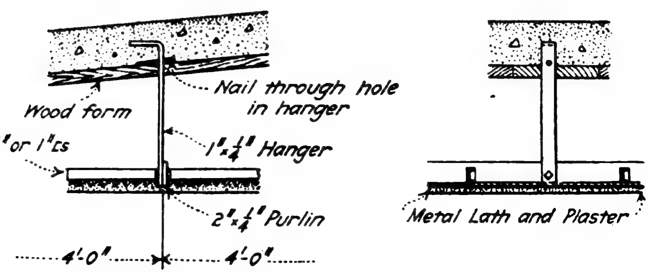


FIG. 142.—Suspended ceiling construction.

generally permits the use of the floor forms without much change and there is the advantage of having all the column heights of the top story the same. Stone-concrete surfacing seems to be the most satisfactory but a cinder mixture is sometimes specified.

If a flat ceiling is not required, or if it is the intention to employ a suspended ceiling (Fig. 142), the roof beams and roof girders may be so inclined as to give the required roof pitch. This method is advantageous when cold weather is likely to overtake the work before it can be closed in, for, with the use of cinders as a filling, there is delay in placing the final roof surface.

Where concrete walls project above the roof proper, grooves or reglets about 1 in. by $1\frac{1}{2}$ in. must be left in the concrete wall in which to insert the edge of the flashing (see Fig. 143). To make a reglet, a strip of wood is nailed to the forms and the strip is taken out when the forms are removed. The flashing is keyed with the reglet, as shown, and cemented up with a rich cement—sometimes rubber cement.

When metal standing flashing is specified on felt roofs, it is necessary to nail through the metal and felt into wooden strips embedded in the concrete slab in order to hold the flashing in place (see Fig. 143). Since expansion and contraction will soon loosen the nails, the nails and the flange of the flashing should be

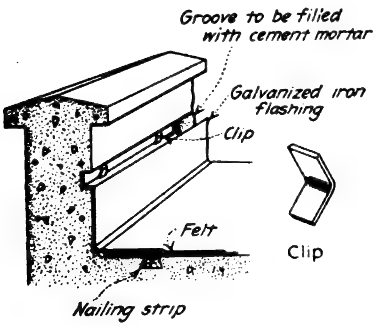


FIG. 143.

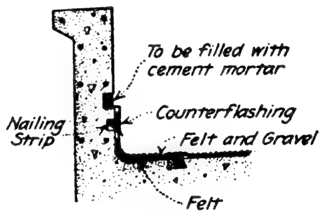


FIG. 144.

covered with a felt strip prior to coating the roof with the top coat of asphalt and gravel. A reinforcement of flax felt, mopped on solidly, appears to be as satisfactory as metal flashing in every way (see Fig. 144). All such felt flashings should be turned up at the parapet walls and curbs at least 4 in. at the highest points of

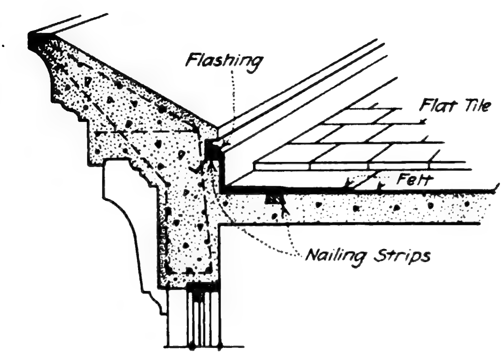


FIG. 145.

the roof, and not less than 12 in. high as the outlets are approached, in order to avoid overflows should the outlets become clogged.

Fig. 143 shows the method employed in applying a galvanized iron flashing for a felt and gravel roof on a large reinforced-con-

crete warehouse. The reglet was made to taper slightly—that is, wider at the outside—in order to permit the easy withdrawal of the wood strip that was nailed to the forms. The groove was made 10 in. above the roof line. When the concrete was being poured for the roof floor, a heavy strip of wood was laid with its

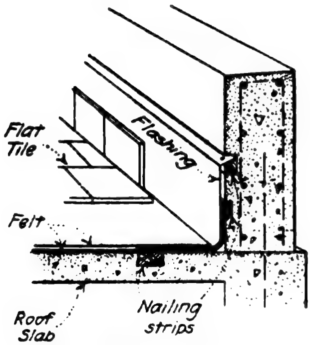


Fig. 146.

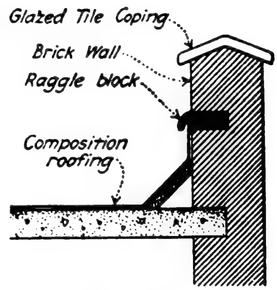


Fig. 147.

upper surface flush with the top surface of the concrete, and 6 in. from, and parallel with, the parapet wall. This strip was employed to nail the edge of the flashing to the roof after the roof had been covered with felt and asphalt. The nails and the

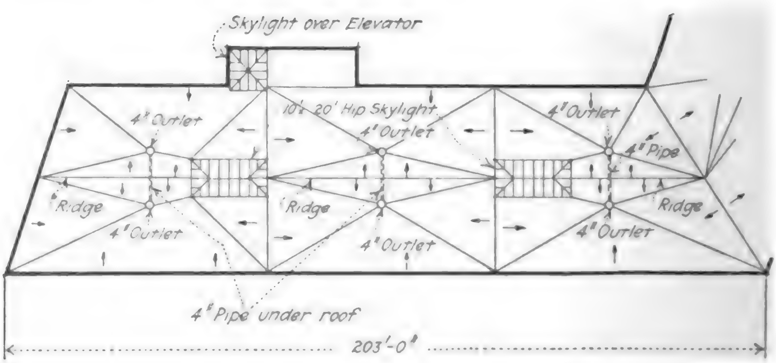


Fig. 148.

flange of the flashing were covered with a felt strip prior to coating the roof with the top coat of asphalt and gravel.

The flashing was secured in the reglets by band-iron clips, as shown. These bands (2 in. long) were bent midway to an angle of about 30°. After insertion they were struck with the hammer

on the bend, which straightened them out and jammed them in place, thereby holding the flashing. The reglet was pointed up with a rich cement to insure a watertight connection. Figs. 144 to 147 inclusive, show different methods of flashing which may be employed. The sketches explain themselves.

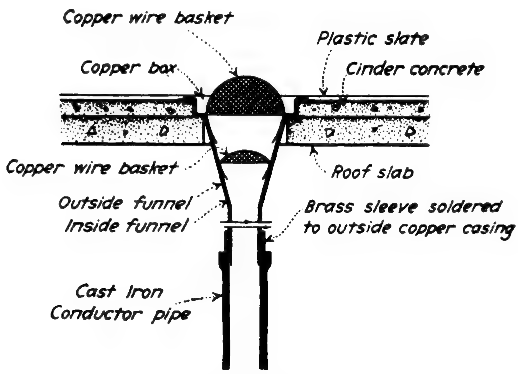


FIG. 149A.

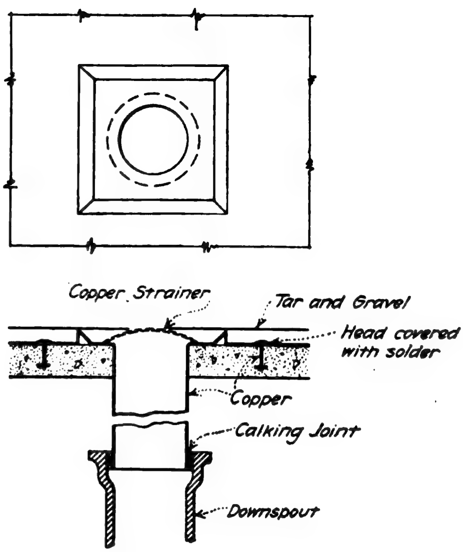


FIG. 149B.

In flat-roof construction with parapet walls, all valleys in the roof surface should lead to drain boxes. Fig. 148 shows the drainage scheme for part of the roof of the Shoe Factory at Cam-

bridge, Mass., built by the Concrete Engineering Co., for The Cambridge Building Trust.

Drain boxes, or conductor boxes as they are usually called, have been made in many ways—lead or copper being generally used. Fig. 149*A* shows a double copper box for conductors used on a factory building for A. M. Lyon, Esq., Roxbury, Mass. Fig. 149*B* is a sketch of a conductor box installed by the Aberthaw Construction Co. in a warehouse for the Portland Savings Bank, Portland, Maine. Wire screens or strainers should always be specified since they prevent the down spouts from becoming

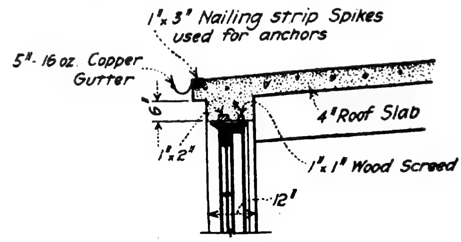


Fig. 150.

clogged by leaves, etc. The down-spouts may be carried down inside or outside the building—depending upon preference and sewer conditions. The conductors should be cast iron when carried inside the building, while corrugated iron, due to its ability to expand without breaking, is better on the outside of the building where freezing of water in the conductors may be expected. All vapor and soil pipes extending through the roof should have an expansion sleeve soldered on and counter-flashed with an inverted copper cone attached to pipe.

Fig. 150 shows the method of draining a roof surface of considerable pitch—a condition under which parapet walls could not be used. This is called hanging gutter construction. The gutters should have a slope of about 1 in. in 16 ft.

The following table will serve as a guide by which to proportion the size of gutters and down-spouts:

Span of roof	Gutter	Conductor
Up to 50 ft.	6 in.	4 in. every 40 ft
50 to 70 ft.	7 in.	5 in. every 40 ft.
70 to 100 ft.	8 in.	6 in. every 40 ft.

There is wide variation in practice in the number of square feet of roof surface to 1 sq. in. of leader opening, but the above table should give some idea of the general practice.

37. **Loading.**—The roof of a reinforced-concrete building should be designed to carry the weight of roof covering and snow which may come upon it. If the roof has considerable pitch, wind pressure should also be considered. A roof load commonly assumed in temperature climates for flat roofs is 40 lb. per square foot, in addition to the weight of the concrete itself.

The snow load varies with the latitude and the humidity. As a maximum it is approximately 30 lb. per horizontal square foot in Canada and northern Wisconsin, 20 lb. in the city of Chicago, 10 lb. in Cincinnati, and rapidly diminishes southward.

The wind load, which acts horizontally, varies with the velocity of the wind. A pressure of 30 lb. per square foot of vertical surface is usually assumed. Several formulas are in existence for determining wind pressure on inclined surfaces. Duchemin's formula which follows, is preferred by many engineers as it is based upon carefully conducted experiments:

$$P = P_1 \frac{2 \sin A}{1 + \sin^2 A}$$

where P = normal pressure of wind in pounds per square foot of inclined surface.

P_1 = pressure of wind in pounds per square foot on a vertical surface.

A = angle of inclination of the roof.

The dead load of any roof may be estimated quite closely from the following data—weights are per square foot of roof surface:

Five-ply felt and gravel roof, 6 lb.

Four-ply felt and gravel roof, 5½ lb.

Three-ply ready roofing, 0.6 to 1 lb.

Slates, $\frac{3}{16}$ in. thick, 7¼ lb.; ¼ in. thick, 9.6 lb. (the common thickness is $\frac{3}{16}$ in. for sizes up to 10 in. by 20 in.).

Shingle clay tiles, 11 to 14 lb.

Spanish tile, 8 lb.

Vitrified roofing tile, 1 in. thick, 9 lb. (including asphalt and five thicknesses of felt, 11½ lb.).

Slate tile, ⅞ to 1 in. thick, 13 lb. (including aphalt and felt, 15½ lb.).

Tin roofing, sheets or shingles, 1 lb.

Copper roofing, sheets, $1\frac{1}{2}$ lb.; tiles $1\frac{3}{4}$ lb.

Corrugated iron, 1 to 3 lb.

Skylights with galvanized-iron frames, $\frac{1}{4}$-in. glass, $4\frac{1}{2}$ lb.; $\frac{5}{16}$ in., 5 lb.; $\frac{3}{8}$ in., 6 lb.

Plaster, 5 lb.

Suspended ceilings, 10 lb.

Cinders, 45 lb. per cubic foot.

Cinder concrete, 112 lb. per cubic foot.

38. Design of Roof Bay.—Problem is to design an interior roof bay of a five-ply felt and gravel roof to support a 30-lb. snow load. Assume a 2-ft. cinder fill with a 2-in. concrete surfacing. Make same assumptions and use same working stresses as in the two-intermediate-beam design of Art. 17. The same column spacing and arrangement of beams and girders will be adopted as in the design mentioned.

The computations given below are without much explanation. The order of design followed is identical with the design of floor bay already referred to in Art. 17 and the student should have no difficulty in understanding the equations and figures given.

Slab.—Total load per square foot exclusive of dead weight of slab is 150 lb., made up of the following items:

five-ply felt and gravel covering	= 6 lb.
2 ft. of cinders	= 90 lb.
2-in. concrete surfacing	= 25 lb.
snow load	= 30 lb.
Total	151 lb., say 150 lb.

A $3\frac{1}{2}$-in. slab ($d = 2\frac{3}{4}$in.) will sustain a load (live plus dead) of 165 (1.2) = 198 lb. per square foot. Load for slab to carry is 150 + 44 = 194 lb. per square foot. $a_s = 0.254$ sq. in. $\frac{3}{8}$-in. round rods — 5 in. on centers. Three $\frac{3}{8}$-in. round rods transversely in each 7-ft. panel.

Cross Beams.—Assume dead load of the stem of beam at 175 lb. per linear foot. Total loading per foot of length = (194) (7) + 175 = 1535 lb.

$$V = \frac{(1535)(21)}{2} = 16{,}000 \text{ lb.}$$

$$M = \frac{(1535)(21)^2(12)}{12} = 677{,}000 \text{ in.-lb.}$$

$$\frac{16{,}100}{105} = 153 \text{ sq. in.}$$

Economical depths for various assumed web widths:

b'	d	$b' \times d$	Depth for shear
8 in.	19.6 in.	157	19.2
9 in.	18.6 in.	167	17.0
10 in.	17.7 in.	177	15.3

The dimensions $b'=10$ in. and $d=19$ in. (total depth 21 in.) will be adopted. This web width is the same as for floors, thus necessitating no change in width of slab forms. The depth of 21 in. is necessary to give a four-rod design. The same arrangement of rods will be employed as is shown in Plate VII except that the bent rods will extend *over* the girder rods. (See discussion concerning this arrangement in Art. 17.)

$$\text{weight of stem} = \frac{(17.5)(10)(150)}{144} = 182 \text{ lb. O.K.}$$

$$b = 10+8(3.5) = 38 \text{ in.}$$

$$\frac{M}{bd^2} = \frac{677{,}000}{(38)(19)^2} = 49.3$$

$$\frac{t}{d} = \frac{3.5}{19} = 0.18$$

$$f_c = 425$$

$$j = 0.92$$

$$a_s = \frac{677{,}000}{(16{,}000)(0.92)(19)} = 2.42 \text{ sq. in.}$$

$4-\frac{7}{8}$-in. round rods.

$$u = \frac{16{,}000}{(4)(2.75)(0.85)(19.5)} = 88 \text{ lb. per square inch.}$$

Since it is proposed to provide thoroughly for diagonal tension by means of vertical stirrups with the bent rods as so much additional web reinforcement, this bond stress will be allowed. (See Arts. 38 and 63, Volume I.)

$$\frac{d'}{d} = \frac{2}{19.5} = 0.103$$

$$p = \frac{2.41}{(10)(19.5)} = 0.0124 = p'$$

$$L = 0.283$$

$$K = 0.0110$$

$$f_c=\frac{677{,}000}{(10)(19.5)^2(0.283)}=628 \text{ lb. per square inch.}$$

$$f_s=\frac{677{,}000}{(10)(19.5)^2(0.0110)}=16{,}200 \text{ lb. per square inch.}$$

$$x_2=\text{or}<\frac{21}{2}\left[1-\sqrt{\frac{(8)(2)}{(12)(4)}}\right]12=53 \text{ in.}$$

$$x_1=\frac{21}{2}-\frac{(40)(10)(0.92)(19)}{1535}=5.95 \text{ ft.}=72 \text{ in.}$$

$(0.012)(19)=0.23$ in. $\frac{3}{8}$-in. round stirrups bent at upper ends O.K.

$$s=\frac{(3)(0.11)(2)(16{,}000)(0.85)(19.5)}{(2)(16{,}100)}=5.4 \text{ in.}$$

At $\frac{l}{8}$ $s=\frac{4}{3}\times 5.4=7.2$ in.

At $\frac{l}{4}$ $s=(2)(5.4)=10.8$ in.

Length for bond of compression rods

$$=\frac{(628)(15)(0.875)}{(4)(80)}=26 \text{ in.}$$

Girder.—Reaction of concentrated loads $=2\times 16{,}100=32{,}200$ lb. Assume dead load of stem at 375 lb. per linear foot. Equivalent total loading per foot of length $=\frac{(32{,}200)(3)}{21}+375$ $=4975$ lb.

$$V=32{,}200+10.5\ (375)=36{,}200 \text{ lb.}$$

$$M=\frac{(4975)(21)^2(12)}{12}(0.90)=1{,}975{,}000 \text{ in.-lb.}$$

$$\frac{36{,}200}{105}=345 \text{ sq. in.}$$

Economical depths for various assumed web widths:

b'	d	$b'\times d$	Depth for shear
12	26.5	318	28.8
14	24.7	346	24.6
15	24.0	360	23.0

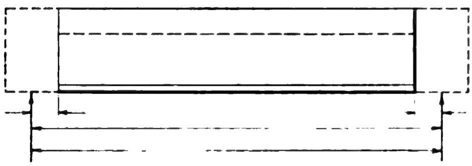
The dimensions $b'=12$ in. and $d=29.5$ in. (total depth 33 in.) will be adopted since greater widths require nearly the same depth to keep f_c within the allowable.

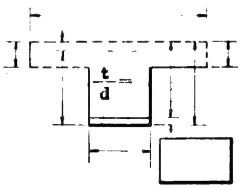
PLATE XII

THE WISCONSIN CONSTRUCTION CO. Sheet No.______

JOB____________________ EST. NO.______ DATE____________________

MADE BY____________ CHECKED BY__________ BEAMS____________________





LOAD (per lin. foot):

Slab =

Wall =

Stem =

Total =

AT LEFT SUPPORT:

$\frac{d'}{d}$ = p = p' =

L = K = M =

f_c = f_s =

u =

AT RIGHT SUPPORT:

$\frac{d'}{d}$ = p = p' =

L = K = M =

f_c = f_s =

u =

CROSS SECTION:

V =

M =

v =

Area for Shear =

Weight of Stem =

$\frac{M}{bd^2}$ =

f_c = j =

a_s =

WEB REINFORCEMENT:

Bent Rods { H Scale / V Scale

Stirrups H. & V. Scale

Moment { H Scale / V Scale

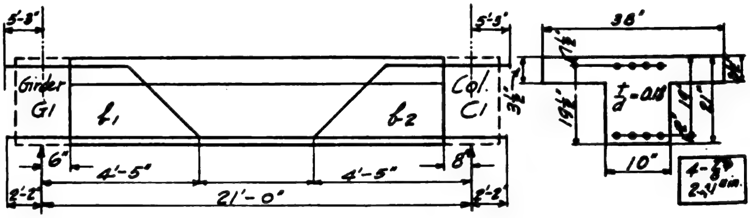
Shear { H Scale / V Scale

PLATE XIII

THE WISCONSIN CONSTRUCTION CO. Sheet No. 1

JOB J. D. Johnson Factory EST. NO. 123 DATE 3/5/1913

MADE BY JMS CHECKED BY WEF BEAMS b1 and b2 Roof



LOAD (per lin. foot):

Slab = (194)(7) = 1358

Wall =

Stem = 175

Total = 1535

AT LEFT SUPPORT:

$\frac{d'}{d} = \frac{2}{19.5} = 0.103$ $p = \frac{2.41}{(10)(19.5)} = 0.0124$ $p' = p$

$L = 0.283$ $K = 0.0110$ $M = 677000$

$f_c = \frac{677000}{(10)(19.5)^2(0.283)} = 628$ $f_s = 16206$

$u = \frac{16100}{(4)(2.75)(0.85)(19.5)} = 88$

AT RIGHT SUPPORT:

$\frac{d'}{d} =$ $p =$ Do $p' =$

$L =$ $K =$ $M =$

$f_c =$ $f_s =$

$u =$

CROSS SECTION:

$V = \frac{(1535)(21)}{2} = 16100$

$M = \frac{(1535)(21)^2(12)}{12} = 677000$

$v =$

Area for Shear $= \frac{16100}{105} = 153$

Weight of Stem $= \frac{(175)(10)(150)}{144} = 182$ O.K.

$\frac{M}{bd^2} = \frac{677000}{(38)(19)^2} = 49.3$

$f_c = 425$ $j = 0.92$

$a_s = \frac{677000}{(16000)(0.92)(19)} = 2.42$

WEB REINFORCEMENT:

$X_2 = \text{or } \angle\ \frac{21}{2}\left[1 - \sqrt{\frac{(8)(2)}{(12)(4)}}\right]12 = 53''$

$X_1 = \frac{21}{2} - \frac{(40)(10)(0.92)(19)}{1535} = 5.95'\ (72'')$

$(0.012)(19) = 0.23''$ ¾" bent O.K.

$s = \frac{(3)(0.11)(2)(16000)(0.85)(19.5)}{(2)(16100)} = 5.4''$

At $\frac{L}{8}$, $s = \frac{4}{3}(5.4) = 7.2''$ At $\frac{L}{4}$, $s = 10.8''$

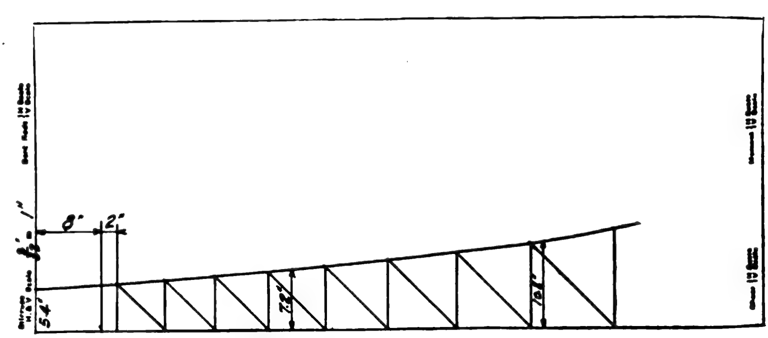
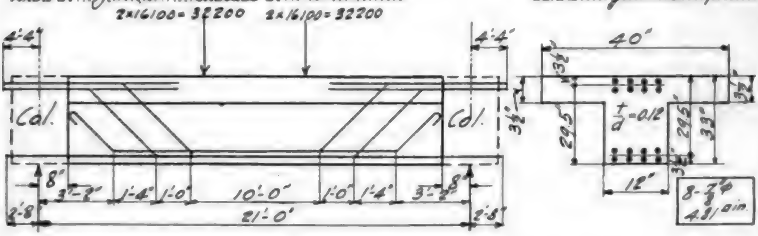


PLATE XIV

THE WISCONSIN CONSTRUCTION CO. Sheet No. 2

JOB J. D. Johnson Factory EST. NO. 123 DATE 3/5/1913

MADE BY J.M.S. CHECKED BY W.E.F. BEAMS 9' - Roof



LOAD (per lin. foot):

Slab=

Wall=

Stem= 375

Total= $\frac{(32200)(9)}{21} + 375 = 4975$

AT LEFT SUPPORT:

$\frac{d'}{d} = \frac{2.5}{29.5} = 0.085$ p= $\frac{4.81}{(12)(29.5)} = 0.0135$ p'= 0.50

L= 0.232 K= 0.0120 M= 1975000

$f_c = \frac{1975000}{(12)(29.5)^2(0.232)} = 750$ f_s= 15800

u= $\frac{36200}{(8)(2.75)(0.85)(29.5)} = 66$

AT RIGHT SUPPORT:

$\frac{d'}{d}$= p= p'=

L= K= Do M=

f_c= f_s=

u=

CROSS SECTION:

V= $32200 + (10.5)(375) = 36200$

M= $\frac{(4975)(21)^2(12)}{12}(0.90) = 1{,}975{,}000$

v=

Area for Shear= $\frac{36200}{105} = 345$

Weight of Stem= $\frac{(12)(29.5)(150)}{144} = 370$

$\frac{M}{bd^2} = \frac{1975000}{(40)(29.5)^2} = 57.0$

f_c= 600 j= 0.95

$a_s = \frac{1976000}{(16000)(0.95)(29.5)} = 4.4$

WEB REINFORCEMENT:

$\frac{V}{jd} = \frac{36200}{(0.85)(29.5)} = 1450$ at support.

$\frac{V}{jd} = \frac{36200-(375)(7)}{(0.95)(29.5)} = 1200$ at third point.

Total diag. tension taken by six rods.

$\frac{1450+1200}{2}(7.0)(12)\frac{2}{3} = 74200$

$\frac{(6)(0.601)(16000)}{0.7} = 82500$

$(0.12)(29.5) = 0.35$ ½"φ Stirrups bent at upper end.

S= $\frac{(2)(0.196)(16000)}{1200} = 5$

$$\text{weight of stem} = \frac{(12)(29.5)(150)}{144} = 370 \text{ lb. O.K.}$$

$$b = 12 + 8(3.5) = 40 \text{ in.}$$

$$\frac{M}{bd^2} = \frac{1,975,000}{(40)(29.5)^2} = 57.0$$

$$\frac{t}{d} = 0.12$$

$$f_c = 600$$

$$j = 0.95$$

$$a_s = \frac{1,975,000}{(16,000)(0.95)(29.5)} = 4.4 \text{ sq. in.}$$

$8 - \frac{7}{8}$-in. round rods.

$$u = \frac{36,200}{(8)(2.759)(0.85)29.5)} = 66 \text{ lb. per square inch.}$$

$$\frac{d'}{d} = \frac{2.5}{29.5} = 0.085$$

$$p = \frac{4.81}{(12)(29.5)} = 0.0136$$

$$p' = \frac{p}{2}$$

$$L = 0.252$$

$$K = 0.0120$$

$$f_c = \frac{1,975,000}{(12)(29.5)^2(0.252)} = 750 \text{ lb. per square inch.}$$

$$f_s = \frac{1,975,000}{(12)(29.5)^2(0.0120)} = 15,800 \text{ lb. per square inch.}$$

At support

$$\frac{V}{jd} = \frac{36,200}{(0.85)(29.5)} = 1450 \text{ lb. per linear inch.}$$

At third point

$$\frac{V}{jd} = \frac{36,200 - (375)(7)}{(0.95)(29.5)} = 1200 \text{ lb. per linear inch.}$$

Total diagonal tension to be taken by six bent rods:

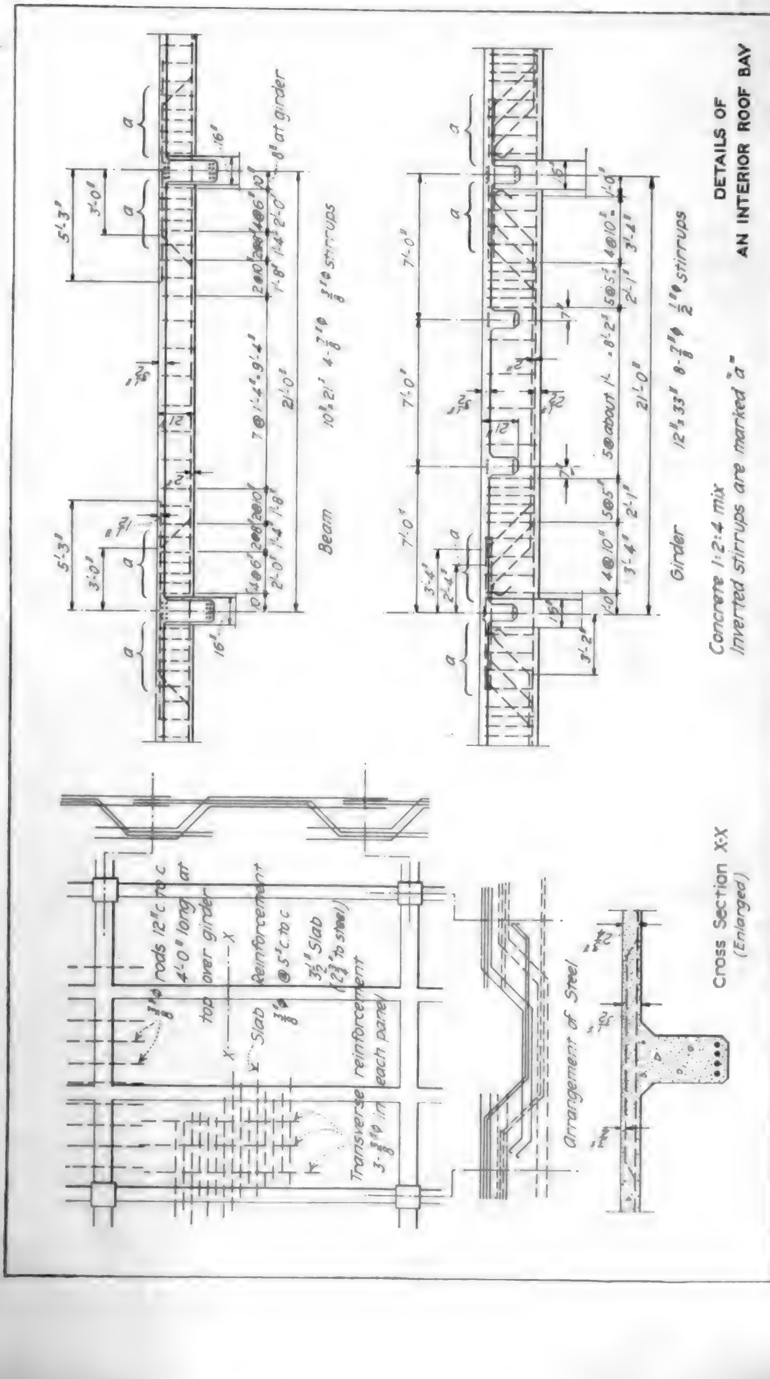
$$\frac{1450 + 1200}{2}(7.0)(12.0)\left(\frac{2}{3}\right) = 74,200 \text{ lb.}$$

Tensile value of bent rods:

$$\frac{(6)(0.601)(16,000)}{0.7} = 82,500 \text{ lb.}$$

$(0.012)(29.5) = 0.35$ in. $\frac{1}{2}$-in. round stirrups bent at upper end O.K.

PLATE XV



DETAILS OF AN INTERIOR ROOF BAY

At third point

$$s=\frac{(2)(0.196)(16{,}000)}{1200}=5 \text{ in.}$$

Length for bond of compression rods

$$=\frac{(750)(15)(0.875)}{(4)(80)}=31 \text{ in.}$$

Plate XII is a suggested form for use in designing beams. Plates XIII and XIV show this form filled out for the above roof beams and roof girder respectively. Plate XV shows the details of the roof bay in question.

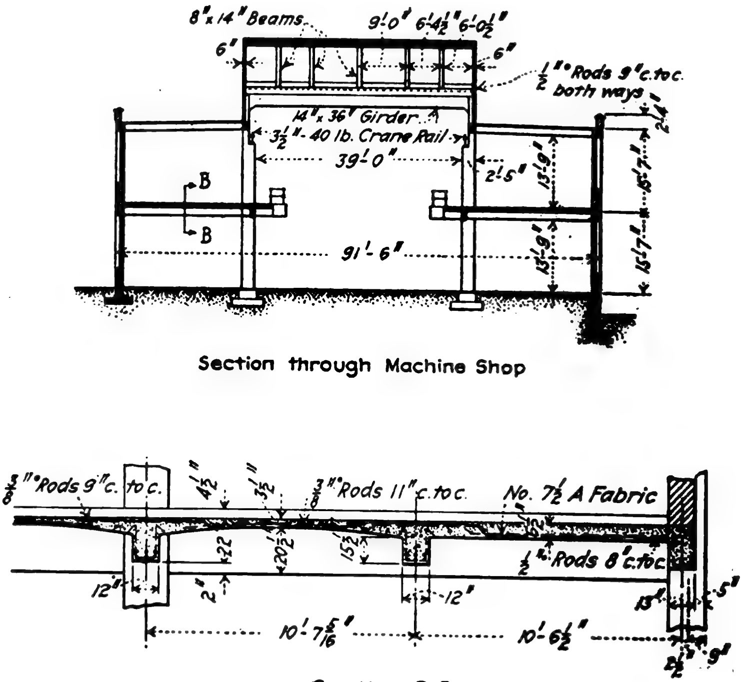


Fig. 151A.

The rectangle shown in Plates XII to XIV inclusive is inserted merely for convenience. It, however, permits all graphical work relating to the location of diagonal-tension reinforcement to be checked up.

39. Saw-tooth Construction.—Saw-tooth roofs, in general, have been found especially well adapted for machine shops and

factories, where it is desirable to have a uniform daylight illumination over the entire floor area. In reinforced concrete, unfortunately, this type of construction is expensive because of the irregularities of the forms, and has been employed only to a limited extent on that account. Typical designs of saw-tooth roofs are shown in Figs. 151, 152 and 153. The skylights are usually arranged to face the north, as the sun's rays would be undesirable and would cause excessive heat in the building in the summer time.

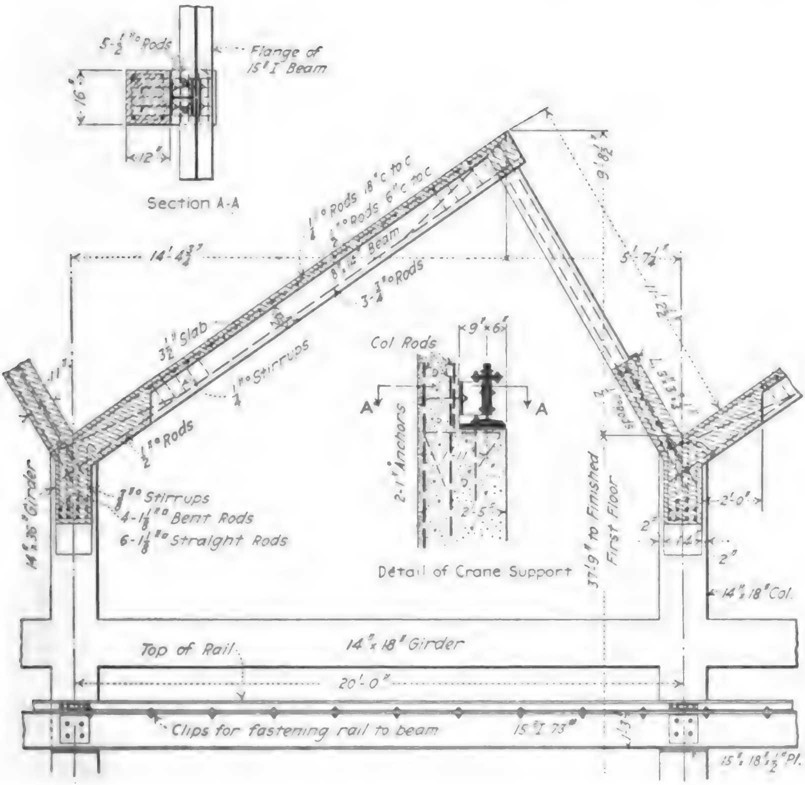


FIG. 151B.

Fig. 153 is a sketch of a saw-tooth roof used in the new Maverick cotton mill (East Boston, Mass.) and designed by Lockwood, Greene & Co. The girders supporting the saw-tooth were made of sufficient stiffness so that no horizontal tie rods were necessary. The lines of columns for the saw-tooth span are 20 ft. center to center and two girders are located in each bay.

The saw-tooth roof construction shown in Fig. 153*A* was designed by Densmore & LeClear, Architects and Engineers, for the A. M. Lyon factory at Roxbury, Mass.

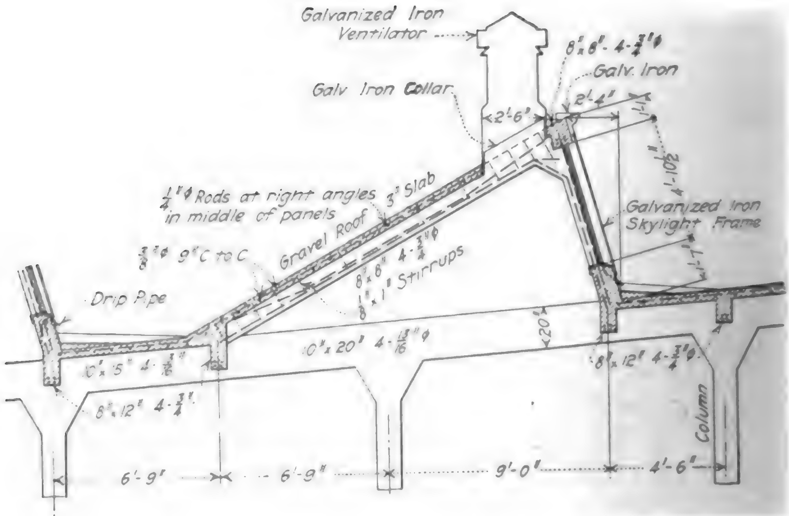


Fig. 152.—Cross-section detail of saw-tooth roof.

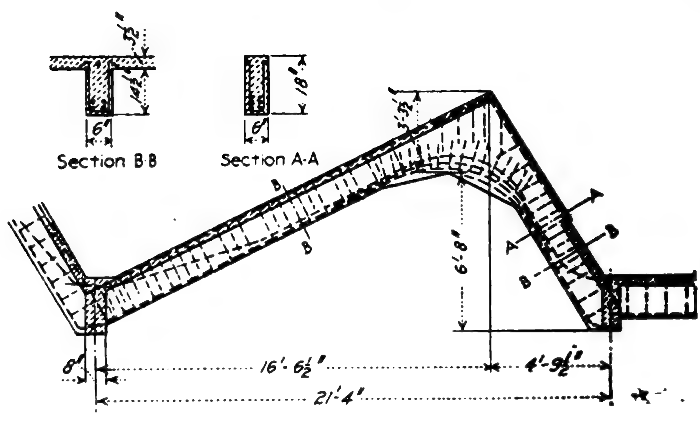


Fig. 153.

A saw-tooth roof construction using separately molded members has been developed by the Unit Construction Co. of St. Louis. Fig. 154 is a cross-section showing the typical arrange-

ment of units. The roof portion of the saw-tooth rests at its lower end on a ledge in the girder and at the upper end on a ledge in the skylight frame. The lower end of the frame also rests on

Courtesy of Densmore & LeClear.

FIG. 153A.—Saw-tooth construction in Roxbury Shoe Factory.

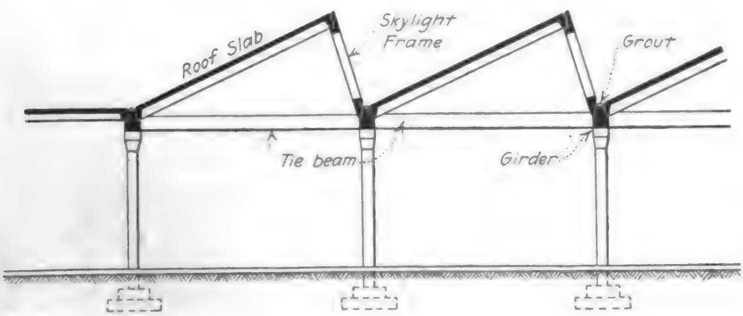


FIG. 154.—Cross-section showing typical arrangement of units in saw-tooth construction, "Unit System."

a ledge in the girder and the horizontal beams are provided in order to tie in the building, and for the possible support of shafting or other installations. The skylight frame extends from column

Courtesy of Universal Portland Cement Co.

FIG. 155.—Saw-tooth construction by the "Unit" method. Factory building for Sturges & Burn Mfg. Co., Bellwood, Ill.

Courtesy of Universal Portland Cement Co.

FIG. 156.—Details of saw-tooth roof, Sturges & Burn Mfg. Co., Bellwood, Ill.

to column and consists essentially of a large flat plate into which the window frames are cast. The connections are made in practically the same manner as the floor connections for this system of unit construction as described in Art. 19.

Saw-tooth roofs of the unit type were employed in the plant of the Sturges & Burn Mfg. Co., Bellwood, Ill. A general view of this plant during construction has been shown in Fig. 76. Figs. 155 and 156 show the details of the saw-tooth roofs.

Saw-tooth skylights are usually glazed with wire glass, held in metallic frames, so that the entire construction is fire-resisting.

Problem

9. Design an interior roof bay of a five-ply felt and gravel roof to support a live load of 30 lb. per square foot. Assume a 1½-ft. cinder fill with a 2-in. concrete surfacing. Column spacing and all other conditions to be the same as in Problem 5. Use designing forms supplied you, and send in computations and roof-bay details.

CHAPTER VII

COLUMNS

40. Details of Design.—Columns exceeding 15 diameters in length should be avoided as very little is known from tests concerning columns of such slender proportions. Fortunately, however, columns longer than 15 diameters are quite rare except, of course, in roof stories where a light roof load often requires very light sections. Where long columns are required, it is better to make the column a little larger rather than use any of the existing formulas which, necessarily, are based entirely upon theoretical considerations. Owing to the difficulty of filling columns of small diameter, an 8-in. column should be the minimum size allowable.

A high percentage of longitudinal reinforcement—say above 4 or 5 per cent—is undesirable on account of the uncertainty of the strength of such columns. In fact, increase in the proportion of cement is usually much to be preferred to a high steel percentage, not only because the cement gives a more definitely-known increase in strength, but also because it is a cheaper column reinforcement than steel. In no case should a leaner concrete mixture than 1:2:4 be used.

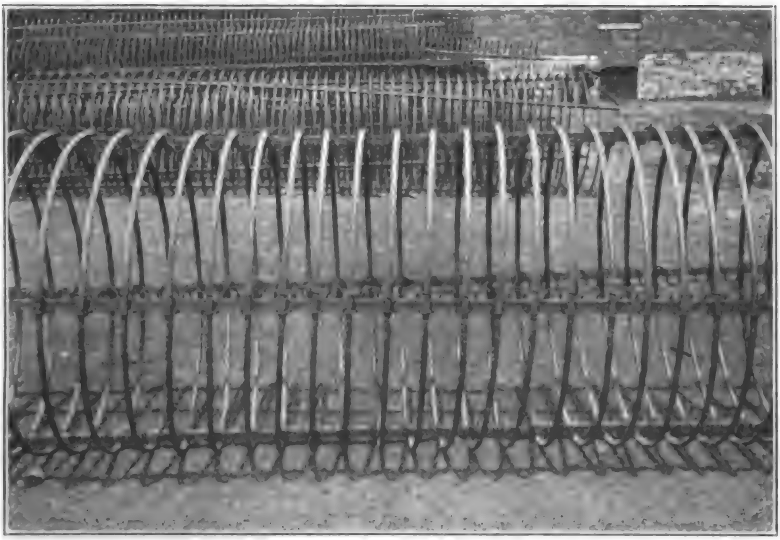
All columns of any considerable length should be reinforced with at least four $\frac{5}{8}$-in. rods, whether or not such reinforcement is theoretically required. This should be done to guard against the possibility of eccentric loading and for safety in construction. The Joint Committee recommends that the length of a plain concrete column be limited to 12 diameters.

The present Building Ordinance of the City of Chicago requires that, where longitudinal rods are not theoretically needed, reinforcing rods shall be used, equivalent to not less than $\frac{1}{2}$ per cent of the cross-sectional area of the column; provided, however, that the total sectional area of the reinforcing steel shall not be less than 1 sq. in., and that no rod or bar be of smaller diameter or of less dimensions than $\frac{1}{2}$ in.

Wherever there is a small eccentricity of column loading,

extra rods should be inserted to take care of the stress due to bending. If, however, the loading is appreciably eccentric, the stresses and amount of reinforcement may be determined as explained in a general way in Chapter IX of Volume I. The method in detail will be given in Chapter XIV of this volume.

If a special fire-proofing is not provided around concrete columns, 1½ in. on all sides should be considered as protective covering and then this thickness should not be included in the calculations for strength. A less thickness than 1½ in. should be sufficient where the contents of a building are not especially inflam-



Courtesy of Ferro-concrete Construction Co.

FIG. 157.—Spiral column reinforcement.

mable. In no case, however, should the steel be nearer the surface than 1½ to 2 in. since it is desirable to prevent any tendency of the vertical rods to buckle under working loads.

It is advisable in all cases to place occasional horizontal hoops around the vertical steel. Although tests do not show that they are absolutely necessary if a 1½-in. to 2-in. concrete covering is provided, yet it is good practice to employ them as an additional precaution against any buckling of the rods under small loads. Such hoops, also, serve to keep the rods in place during the pouring of the concrete and should not be farther apart than 18

in. nor exceeding about 20 times the diameter of the vertical rods. For columns of moderate size, ¼-in. wire hoops are generally used 12 in. on centers.

Bands, hoops or spirals (Fig. 157) to be considered effective should have a clear spacing not greater than one-sixth (preferably one-tenth) the diameter of the enclosed column—in no case, however, more than 2½ in.—and should be equal in amount to at least 1 per cent of the volume of the column inside the hoops. (See second report of the Joint Committee given in the Appendix.) It is always better to use longitudinal rods in connection with hooping, as the use of lateral reinforcement alone does not seem altogether advisable. It should be noted that the Joint Committee recommends an increase in the allowable working stress on the concrete of hooped columns only when the ratio of the unsupported length of column to the diameter of the hooped core is not greater than 8.

It should always be kept in mind that hoops and spirals are tension reinforcement and subject to all the rules governing the design of such steel. If hoops are used, the ends should lap a sufficient distance to secure the requisite grip, or else the ends should be bent toward the center of the column to accomplish the same purpose. When spiral reinforcement is employed, it is important to have one continuous piece from top to bottom, or else joints should be made as just explained.

Hooping should extend from the bottom of the column to the under side of the slab above and adequate means should be provided to hold it rigidly in place so as to form a column, the core of which will be straight and well centered. For ordinary percentages of hooping, tests show that wire of high-carbon steel gives a much larger increase in the ultimate strength of column than wire of mild steel.

The core of a hooped column should preferably be circular in cross-section, but square and rectangular hoops are sometimes employed in practice (the Joint Committee recommends that only circular hoops be employed). In such cases, the hoops are probably less effective, but it is not known how much. The ideal condition, theoretically, is hooped columns of circular cross-section with circular cores. The expense incidental to the use of circular forms, however, is quite likely to be prohibitive. Columns are commonly made square or octagonal in section with the circular form retained for the hoops.

When light vertical rods are used they may be spliced by lapping a sufficient distance above the floor level (about 30 diameters) to develop the requisite bond strength. They should be bent slightly inward at the top in order to come just within the rods of the column above, thus preventing any tendency of the load coming down from the upper rods to bulge the column. Rods much over an inch in diameter should be milled square and butted together in a secure manner. Some authorities advise this method of splicing for all rods except those which are not figured as taking stress.

The splicing of large rods may be effected by joining them in rather closely fitting pipe-sleeves. All such splices should be made just above the floor level and not more than 12 in. above the same. Since each rod should find a bearing on a rod below, the number of rods increases downward in the building and, unless the loading is eccentric, great care should be taken to have the rods symmetrically arranged in each story. Rods may extend through several stories if so desired, but it is difficult to hold them in place while concreting the lower lengths.

At the bottom of the columns, the loads from the rods should be distributed over the footing by means of a steel bearing plate. In order not to overstress the concrete in the column, the concrete immediately below the plate should be enlarged so as to bring the average pressure within the allowable. Tests described in Art. 69 of Volume I show that if considerable lengths of the longitudinal reinforcing rods are bent outward into a reinforced-concrete footing, the strength of column will be about as great as when these rods are bedded on a metal plate. The results, however, also show that the use of metal plates leads to greater uniformity and strength. Base plates should be set in a thin grout of proportions about 1 : 2 in order to prevent hollow places under the plates.

In our modern city buildings where the floor space on the first floors is valuable, the use of a steel column core (Figs. 158 and 159) presents a very economical and efficient construction. To be able to count upon the concrete in such columns the steel itself should be sufficiently rigid to act as a column and the concrete should be well enclosed either by the steel form itself or by means of bands or hooping. However, if the amount of steel is very large, the relative value of the concrete is more uncertain, and its element of strength should be neglected. Structural steel reinforcement

is sometimes employed in the form of a cross in the center of the column or, more often, channels are employed connected by riveted latticing. The best connection between column and beam is a structural steel seat riveted to the column, and of sufficient size to carry the entire load in bearing These seats do not appear in the finished surface of rooms and their stiffness is considerably assisted by the concrete shell.

Courtesy of Turner Construction Co.

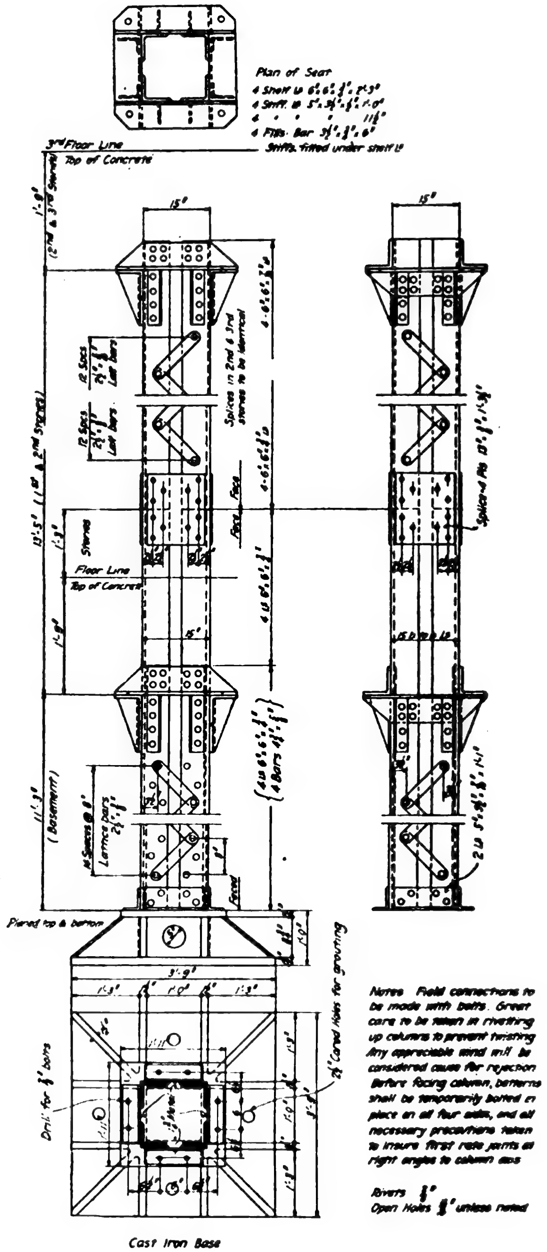
FIG. 158.—Arbuckle Brother's Warehouse, Brooklyn, N. Y. Structural steel columns used in the first three floors to economize floor space and carry the very heavy floor loads used in this building.

Where structural steel columns are used, the floor beams and girders should be figured as simply supported unless the usual continuity rods are run through the column. The structural steel and concrete column shown in Fig. 160 was designed by the Aberthaw Construction Co. of Boston for the Storage Building of the Winchester Repeating Arms Co. It is a typical design of this type of column.

Wherever concrete walls are to be built against concrete columns at some later time, weather breaks or keyways should be formed

Courtesy of Aberthaw Construction Co.

FIG. 159.—Mushroom floor with steel columns. Massachussetts Cotton Mills, Lowell, Mass.



Plan of Seat
3rd Floor Line
Top of Concrete
Floor Line
Top of Concrete
Cast Iron Base
Notes. Field connections to be made with bolts. Great care to be taken in rivetting up columns to prevent twisting. Any appreciable wind will be considered cause for rejection. Before facing column, battens shall be temporarily bolted in place on all four sides, and all necessary precautions taken to insure first rate joints at right angles to column axis.

FIG. 160.

in the columns. This is accomplished by nailing small strips on the inside of the forms. Where windows are to be set into column reveals, window ties should be embedded. In order to obtain a low cost for form work it is better to vary square columns in one dimension rather than in two, both on account of the columns themselves and the beams framing into them.

Cinder-concrete cylinders have been used to some extent as a protective covering for reinforced-concrete columns and also to take the place of column forms. In Factory No. 1 of the plant of the Bush Terminal Company, South Brooklyn, New York, the interior columns are cylindrical and composed of an outside shell of cinder concrete 2½ in. thick. These cinder-concrete cylinders were prepared in advance in 2-ft. lengths, in a zinc mold, with spiral hooping and expanded metal forming the inner surface. After hardening, they were set one upon another in the building, and filled with concrete.

Separately molded columns are described in Art. 19. Typical details of such columns are also shown.

41. Loading.—Column loading has already been considered to some extent in Art. 22.

In addition to its own dead weight, a column should be designed to carry the live and dead loads of the roof and floors above. The full live load on floors must be used in designing columns for warehouses or buildings for heavy mercantile and manufacturing purposes. In other buildings, however, where it seems reasonable to suppose that a full live load will never occur on all floors simultaneously, reductions should be made as follows:

For columns carrying	Reduce total live load
Two floors	15 per cent.
Three floors	20 per cent.
Four floors	25 per cent.
Five floors	30 per cent.
Six floors	35 per cent.
Seven floors	40 per cent.
Eight floors	45 per cent.
Nine or more floors	50 per cent.

42. Column Brackets.—Column brackets, such as are shown in Fig. 161, are serviceable in stiffening the building frame and in decreasing the stress in the girders. They should not, however, be considered as decreasing the girder span, but simply as so much additional protection against failure.

15

Brackets are usually required in high buildings in order to brace the frame properly against wind stresses. If, however, buildings are both high and narrow, the stresses in the members due to wind

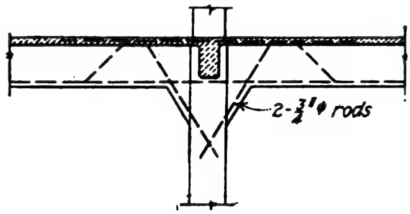
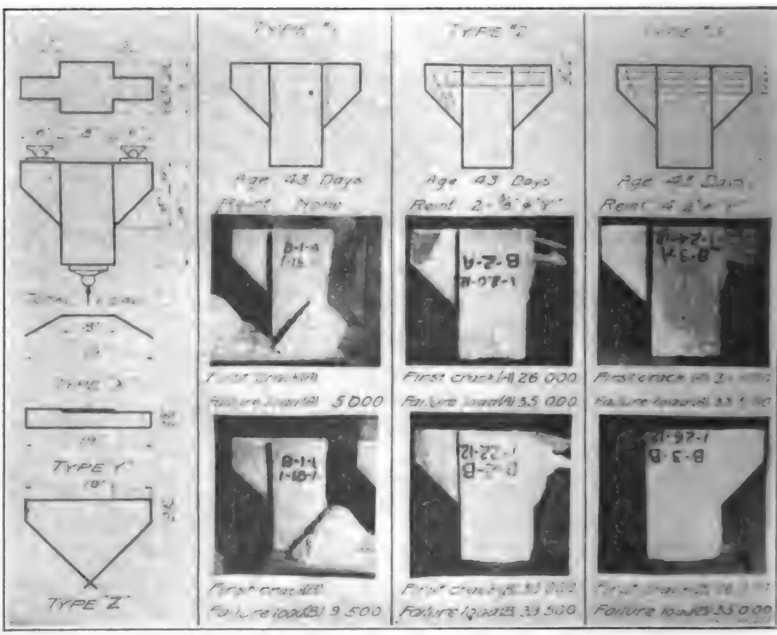


FIG. 161.

should be figured and proper provision made for the same. The method of computing such stresses will be explained in Art. 75.

A column supporting a roof may also serve as a crane post or

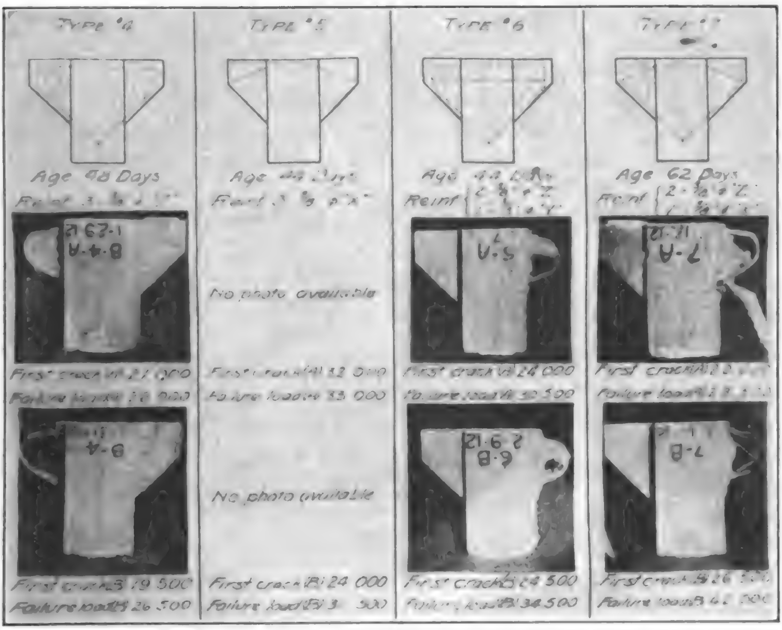


Courtesy of Unit Construction Co.

FIG. 162.—Data on bracket tests.

carry a heavy load on a bracket. The method of computing the bending moment due to such eccentric loading will be explained in chapter XIV, Art. 73.

In order to develop and demonstrate unit construction methods, the Unit Construction Co. of St. Louis has equipped a laboratory with a 50,000-lb. testing machine and other necessary apparatus. Among the experiments that have been made are a number that were devised to throw some light on the subject of bracket design. The following description of these tests has been taken from a paper presented by Mr. John E. Conzelman, Chief Engineer of the Unit Construction Co., before the National Association of Cement Users, Pittsburg, January, 1913:



Courtesy of Unit Construction Co.

FIG. 163.—Data on bracket tests.

"The dimensions of the specimens are shown in Fig. 162. The concrete was a 1 : 2 : 4 mix, the aggregate being Meramec River sand and gravel. The specimens were tested in an inverted position on account of greater ease in handling. The load was applied through a hardened-steel ball, bearing in a cup-shaped depression in a heavy steel plate. The brackets rested on 1-in. × 4-in. steel plates carried on rollers as indicated in sketch. Even bearing between plates and concrete was assured by means of plaster-of-Paris joints. Seven types

of brackets were tested; type No. 1 being without reinforcement—the others reinforced as shown.

"Three types of reinforcement were used, singly and in combination. Type *X* is a short length of bar bent downward at each end; type *Y* consists of a single bar bent in the shape of a rectangle; type *Z* consists of a single bar bent to follow the shape of the bracket. The type *Z* reinforcement is as you know, the type usually employed.

"The specimens were tested at the age of six weeks as near as possible, except type No. 7 specimens, which were tested at the age of 62 days. Two specimens of each type were tested, and the photographs of Figs. 162 and 163 show clearly the condition of the specimens after failure. The load causing the first noticeable crack, and the load causing failure are also shown. The position and extent of the cracks are shown in the sketch of each type.

"As was to be expected, none of the specimens failed through tension in the steel. The failures were due to shearing or diagonal-tensile stresses, and insufficient bond. The action is similar to that which exists at the end of a beam; the condition is aggravated, however, by the difficulty of securing sufficient bond to develop high stresses in the steel.

"Although the conclusions that are derived from these tests are not final, yet the fact is brought out that the type of bracket ordinarily used is the least efficient of the forms tested. The failure is fairly sudden and the concrete is almost completely destroyed. The most satisfactory form is type *Y*, or closed loop. With a number of loops spaced close together the failure is slow and even after the concrete is partially destroyed considerable load-carrying capacity exists. This feature is shown by the photographs of type No. 3.

"The average shear per square inch on the vertical plane of junction with the column at first load and at failure is given in the following table:

Type	Shear at first crack	Shear at failure
1	100 lb. per sq. in.	100 lb. per sq. in.
2	400 lb. per sq. in.	530 lb. per sq. in.
3	410 lb. per sq. in.	490 lb. per sq. in.
4	285 lb. per sq. in.	375 lb. per sq. in.
5	400 lb. per sq. in.	460 lb. per sq. in.
6	345 lb. per sq. in.	465 lb. per sq. in.
7	330 lb. per sq. in.	470 lb. per sq. in.

"A study of these tests leads to the conclusion that the most efficient reinforcement would be the *Y*-type loop bars, some of which would

be bent downward into the bracket. Another series of tests to determine this point is now being made."

43. Design of Typical Interior Column for Three-story Building.—In order to show in detail the method of column design, the computations will be given for the design of a typical interior column of a three-story building. The columns will be spaced 21 ft. on centers both ways and the two-intermediate beam design of Plate V will be adopted as the typical floor-bay design for all three floors. The design shown in Plates XIII, XIV, and XV will be adopted for the typical roof bay.

The height from finished floor to finished floor will be made 13 ft., except the basement which will be made 11 ft. A concrete mixture of 1 : $1\frac{1}{2}$: 3 will be used, which will allow a working stress of 560 lb. per square inch when longitudinal steel only is employed and 810 lb. per square inch when effective hooping is provided. As recommended in the second report of the Joint Committee, the concrete will be assumed to have a modulus of elasticity one-twelfth of that of the steel. The size of octagonal columns will be given in terms of the short diameter.

The load coming from the roof may be found by multiplying the total shear on both girder and cross-beam by 2, and adding; or

$$(2)(36{,}200)+(2)(16{,}100)=104{,}600 \text{ lb.}$$

Similarly, each floor load is

$$(2)(55{,}300)+2(25{,}000)=160{,}600 \text{ lb.}$$

The building in question will be considered as for manufacturing purposes and no reduction in live load will be made. The complete design of column is given on page 216. Tables 12 and 13 of Volume I were used in the design.

The recommendations of the Joint Committee have been followed and the amount of vertical steel is in every case less than 4 per cent and more than 1 per cent. The pitch of the spiral hooping is given to the nearest $\frac{1}{8}$ in., as the mills that fabricate spirals can manufacture them to such a pitch. The weight of column has been figured for a length 2 ft. less than the story height in order to compensate somewhat for taking the weight of beams and girders as extending from center to center of supports. It is the intention to splice all rods, not lapped, by means of a tightly-fitting pipe sleeve about 12 in. long.

COLUMN SCHEDULE

Story	Kind of load	Amount of load	Shape and size of column	Vertical steel (rounds)	Spiral (core 3 in. less than dia. of column)
Third	Roof	104,600	16 in.	8–$\frac{7}{8}$	
	Column	2,500	octagonal		
	Total	107,100			
Second	Floor	160,600	21 in.	12–$\frac{7}{8}$ (lap	dia. $=\frac{3}{8}$ in.
	Column	4,200	octagonal	above 27 in.)	pitch $= 2\frac{3}{4}$ in.
	Total	271,900			
First	Floor	160,600	26 in.	8–$1\frac{1}{8}$	dia. $=\frac{1}{2}$ in.
	Column	6,500	octagonal	8–$\frac{7}{8}$ (lap above 27 in.)	pitch $=3\frac{3}{4}$ in.
	Total	439,000			
Basement	Floor	160,600	30 in.	16–$1\frac{1}{8}$	dia. $=\frac{1}{2}$ in.
	Column	7,000	octagonal		pitch $=2\frac{1}{4}$ in.
	Total	606,600			

PROBLEM

10. Make a complete design of a typical interior column for a three-story building using the floor and roof loads determined from Problems 5 and 9. Follow the recommendations of the Joint Committee throughout. Take the same story heights as given in the above design.

CHAPTER VIII

FOUNDATIONS

Reinforced concrete is well adapted to the construction of foundations. As compared with plain concrete, its advantages for spread footings are a reduction in the amount of excavation required, a saving in material, and a reduction in the weight of the foundation itself.

44. Bearing Capacity of Soils.—A detailed discussion of the bearing capacity of different soils is not within the province of this treatise, but a few general remarks on this important subject may prove of some value.

The bearing capacity of soils depends upon their composition, the degree to which they are confined, and the amount of moisture which they contain. An approximate idea of the loads which may be safely placed upon uniform strata of considerable thickness may be obtained from the table in Art. 5.

There are many kinds and mixtures of soils and it is not always possible to judge the safe bearing capacity of any given soil by reference to a table such as that above mentioned. When there is any doubt, tests or borings should be made. After opening the trenches, the best method is to load known areas and observe the settlement, but, in interpreting the results, the fact should not be overlooked that a small area will bear a larger load per unit of area for a short time than a larger area will perpetually. Thus, the area tested should be as large as practicable and the test should continue for some time.

Ordinary soils will usually bear more weight the greater the depth, owing to the fact that they become more condensed from the superincumbent weight. With clay, depth is especially important as there is less liability of its being displaced laterally due to other excavations in the immediate vicinity, and also because at greater depths the amount of moisture in it is not subject to so much variation. In any soil, the bed of the foundation should be below the reach of frost.

It is safe to say that any rock in its native bed will bear the heaviest load that can be brought upon it by any masonry construction. It scarcely ever happens that rock is loaded with the full amount of weight which it is capable of sustaining. In preparing a rock bed, all loose and decayed portions should be cut away and the bed dressed to a plane surface as nearly perpendicular to the direction of the pressure as is practicable. Any fissures or seams in the rock should be filled with concrete. A sloping surface should be stepped or the foundation designed with sufficient toe to prevent sliding.

Sand when dry, or wet sand when prevented from spreading laterally, forms one of the best beds for a foundation. However, porous, sandy soils are easily removed by running water and require extreme care at the hands of the constructor.

Piles are used in such soils as are not able to bear the weight of structures without an excessive spread of the footing base. If a pile is driven so that its lower end rests upon a hard stratum, the loading is limited by the strength of the pile considered as a column. The load on an ordinary bearing pile is carried by the friction of the earth on the sides of the pile. The only formulas in anything like general use for friction piles are the following, known as the Engineering News formulas:

$$\text{For a pile driven with a drop hammer, } P=\frac{2Wh}{s+1}$$

$$\text{For a pile driven with a steam hammer, } P=\frac{2Wh}{s+0.1}$$

in which P is the safe load in pounds, W the weight of the hammer in pounds, h the fall of the hammer in feet, and s the penetration or sinking in inches under the last blow, assumed to be sensible and at an approximately uniform rate. The above formulas were deduced for wood piles, but they are the best there are for concrete piles. They are also claimed to be safe, for ordinary weights of hammer and the usual height of fall, for a pile that acts as a column.

In the driving of piles, Mr. Joseph R. Worcester, consulting engineer, of Boston, advises for piles which meet a hard resistance, a penetration of 1 in. under a 2000-lb. hammer falling 10 ft.; and for piles held by friction a penetration of 3 in. under a 2000-lb. hammer falling 15 ft. Ordinary piles of spruce and Norway pine will usually sustain 10 tons by friction and 15 tons in bearing.

These piles should never be less than 6 in. in diameter at the small end and never more than 18 in. at the large end. They may be driven 2 to 3 ft. apart depending upon their length, the hardness of the soil and the size of the butts. It has been found that little or no additional bearing power is secured if the spacing is much less than 2 ft. on centers. Concrete piles are manufactured in different sizes and shapes, and their bearing capacity should be thoroughly considered for each case.

45. Pressure on the Soil.—A column or wall footing must be spread until the safe bearing capacity of the soil is not exceeded. An effort need not be made to eliminate all settlement, but rather to so plan the building that whatever settlement does take place will be uniform. In other words, to prevent cracks in a building, the center of the loads from the columns or other portions of the structure should coincide with the center of gravity of the base, and the unit pressures under the footings should be such that uniform settlement will result.

In buildings subject to shock or constant live load, the area of the footings should be proportioned for the full live and dead loads. In other buildings, that footing should be chosen in which the live load bears the highest percentage to dead load and its area determined for the total load at the allowable bearing in the soil; the pressure of the *dead* load per unit area should then be determined and the area of all other footings should be proportioned for dead load only with this unit pressure. Of course, the preceding statement applies without change only when the soil is uniform throughout.

The pressure on the foundation of a tall building should be considerably less than that which may be allowed on the foundation of a low one-story structure. In the former case a slight inequality of bearing power, and consequent unequal settling, might imperil the safety of the entire building; while in the latter case no serious harm would result.

46. Types of Column Footings.—Foundations for columns may be divided into three principal forms: single footings; combined footings, including two or more columns; and raft foundations covering the whole foundation area.

47. Single Column Footings.—A single symmetrical slab is the most common form of spread footing. Two illustrative problems will serve to show the different methods of design.

Design of Footing for Interior Column

Design a single square footing for the column of Art. 43 (octagon 30-in. short diameter) carrying 610,000 lb. Consider the safe bearing capacity of the soil at 2½ tons per square foot. Use a 2000-lb. concrete, with medium steel reinforcement.

It will be assumed that good soil may be found at the required depth below the basement floor. A base plate will be provided under the column rods, and this plate will be placed on top of the finished footing. The top of the footing will be made with an area about twice that of the column and a bearing pressure of 650 lb. per square inch will be permitted. (See Arts. 10 and 40, Volume I.) Investigation shows that this stress is but slightly exceeded, and thus the base of the column need not be enlarged on this account. If desired, the base plate may be placed some distance down in the footing, but under such an arrangement the column rods for the first story must be placed while the footing is being poured, which is troublesome. Sometimes, however, this is avoided, especially where the footing is at considerable depth, by inserting only short column rods of such a length that they may be properly spliced immediately above the basement floor. If this is done, the first floor can be laid entire and the first-story columns started above it.

The top of the footing will be made square, 42 in. on a side. This provides a 6-in. ledge all around so that, if it is desired to erect the first-story columns before pouring the basement floor, the column forms will have some place on which to rest. In computing stresses in the footing, the column may be assumed to have a 30-in. square section, without any appreciable error. The dead weight of the footing will be taken at 45,000 lb.

The required area of footing is found by dividing the total load by the safe bearing capacity of the soil.

$$\frac{655{,}000}{(2.5)(2000)} = 131 \text{ sq. ft.}$$

We shall select an area 11 ft. 6 in. square.

A common construction for single column footings is shown in Fig. 164. After deducting the area of the column base, the remainder of the footing slab is considered as eight cantilevers, four running parallel to the sides and four on the diagonals.

Area of assumed 30-in. square column = 6 sq. ft. approximately.

$$132 - 6 = 126 \text{ sq. ft., net area.}$$

The pressure on one-fourth of this net area—that is, on area *ABCD*, Fig. 164—may be assumed to be carried to the line *BC* where the bending moment will be a maximum. The distance out from *BC* to the center of gravity of the area *ABCD*—distance x, Fig. 165—may be found by the following formula:

$$x = \frac{\frac{ac}{2} + \frac{2}{3}c^2}{a+c}$$

which the student can easily verify. In the problem at hand, $x = 2.74$ ft.

It will be assumed that sufficient accuracy may be obtained if we consider the entire load of $2\frac{1}{2}$ tons per square foot as causing

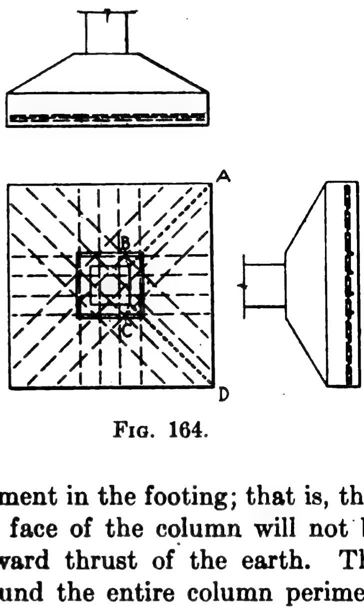


FIG. 164.

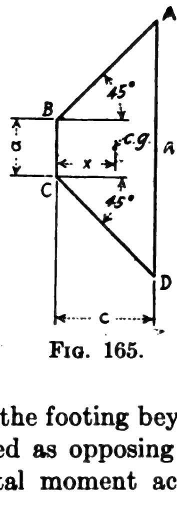


FIG. 165.

moment in the footing; that is, the weight of the footing beyond the face of the column will not be considered as opposing the upward thrust of the earth. Thus, the total moment acting around the entire column perimeter is

$$(126)(2.5)(2000)(2.74)(12) = 20{,}700{,}00 \text{ in.-lb.}$$

With $f_s = 16{,}000$, $f_c = 650$, and $n = 15$, Table 2, Volume I, shows $p = 0.0077$ and $K = 107.4$

$$M = Kbd^2 \text{ or } d = \sqrt{\frac{M}{Kb}} = \sqrt{\frac{20{,}700{,}000}{(107.4)(30)(4)}} = 40 \text{ in.}$$

We shall try $d = 40$ in. (least depth for any set)

$$a_s = pbd = (0.0077)(30)(4)(40) = 37 \text{ sq. in.}$$

This total amount of steel will be divided by eight in order to determine the amount of steel in each band. If desired, more rods can be placed parallel to the sides of the footing than in a diagonal direction. In this design six 1-in. round rods will compose each set of rods.

In view of the footing tests made by Prof. Talbot and referred to in Art. 18, it would seem that the rods need not all pass directly under the column. In fact, if the width of bands needs to be increased in order to cover the entire area of the footing, it would be conservative to say that this may be done provided that the increase in the width of each band is not great. Until more tests have been made along this line, it would seem wise (especially in footings in which the depth decreases toward the outer edge) not to be too radical in the design of such important structures as footings. Where a small increase in the width of bands does not suffice to cover the entire area of footing, a few short cross-rods may be employed to span the open spaces. Increase in band width should preferably be made in the bands parallel to to the sides of the footing as, in these bands, the lengths of the rods do not change.

It should be noticed that the diagonal rods have a greater length subjected to cantilever action than the rods parallel to the sides of the footing. Since it is difficult to calculate accurately the stresses in a square footing, it does not seem proper to attempt any but equal division of moment between the rods in the different directions. Some designers prefer an octagonal shape of footing so that all rods will have approximately the same length. The rods should be cut from 2 to 4 in. shorter than the total width of footing, but if desired, the rods may be cut considerably shorter and staggered so as to allow for the decrease in bending moment from the column toward the edges of the footing.

The above method of finding stresses due to moment may be called the *cantilever* method—that is, the footing is figured as a free cantilever. It should be clear that the resulting stresses in a symmetrical footing computed by this method are higher than actually exist since the resistance in a circumferential direction is disregarded. (See Art. 18.)

The thickness of the footing may be decreased by judgment toward the edges without reducing the effective strength. In the problem at hand, the depth at the outer edge will be made 12 in.

The footing tests described in connection with the discussion of flat-slab floors show that, as an indication of the tendency to diagonal tension, the intensity of shearing stress should be computed on a section at a distance out from the face of column or pier equal to the depth of the footing to the steel.

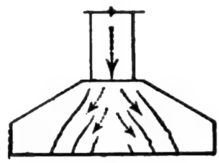


Fig. 166.

This fact does not appear strange when it is considered that column action or diagonal compression exists for some distance from the face of the pier. For example, if cracks should tend to form as shown in Fig. 166, the short columns between the cracks would be in position to receive direct compression. As we know, these cracks make an angle of approximately 45° with the horizontal. A somewhat similar condition to this exists at the supports of flat-slab floors. If properly designed for continuity, continuous beams at the inner supports (and the end supports,

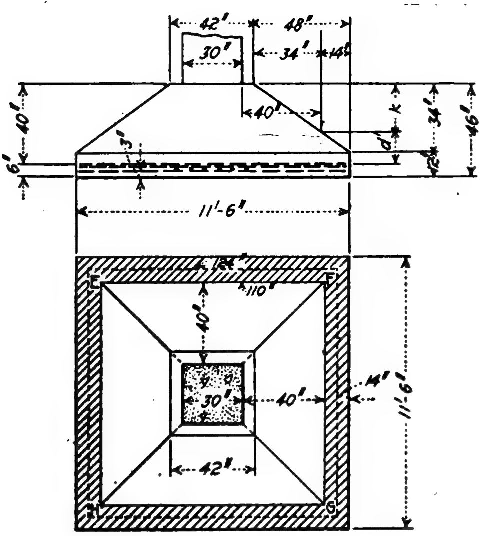


Fig. 167.

if fixed) might also be included, but the ratio of span of beam to depth is much less than in flat-slab floors, and the point of inflection is unlikely to be so close to support as to affect any application of the above principle in the finding of the proper section to measure diagonal tension. Until we have more

experimental data along this line, the placing of stirrups close to the supports of continuous beams should continue and the maximum shear should be used in the computations for stirrup spacing at these points.

Referring to Fig. 167, it should be clear that the footing in question should be figured for shear (as measuring diagonal tension) on a vertical plane through the footing at a distance of 40 in. from the face of the column—as in *EF*, for example. The reaction of the soil on the cross-hatched area is what causes shear on the vertical planes through *EFGH*.

$$d' = 40 - k = 40 - \frac{(34)^2}{48} = 16 \text{ in.}$$

$$\text{Total shear on } EF = V = \frac{(124)(14)(2.5)(2000)}{144} = 60{,}000 \text{ lb.}$$

$$v = \frac{60{,}000}{(110)(0.875)(16)} = 39 \text{ lb. per square inch.}$$

Thus the footing as designed needs no stirrups.

Whenever the unit shear is found too great at the given distance out from the face of column, the plane where the shear is 40 lb. per square inch should be found by trial. Then the area requiring stirrups may be easily determined. Two-thirds of the unit shear on *EFGH* would be carried by the steel, and this amount multiplied by one-half the area requiring stirrups would give the total amount of tension for which to provide web reinforcement. This total tension divided by the tensile value of one stirrup would give the number of stirrups required.

In nearly all cases a slight deepening of the footing at the outer edge is all that is necessary to avoid using stirrups. The placing of stirrups is troublesome and footings should be designed so that web reinforcement will not be needed, if this can be done without greatly increasing the dimensions.

The bond stress along the horizontal tension rods must now be investigated. This should be determined on a plane passing through the face of the column and caused by the load on the footing area exclusive of the area directly below the column.

$$u = \frac{V}{\Sigma ojd} = \frac{(126)(2.5)(2000)}{(48)(3.14)(0.875)(40)} = 120 \text{ lb. per square inch.}$$

Deformed bars are advantageous in footings because of increased bond strength. A good type of deformed bar will carry 150 lb. per square inch on a 2000-lb. concrete. If desired, 13–$\frac{3}{4}$-in. plain

round rods could be employed in each direction in the footing design under discussion, but this arrangement gives a great deal more steel than is needed for moment.

The punching shear on a column footing, on an area equal to the perimeter of the column times the depth to the steel, should not exceed the allowable value. In the problem at hand:

$$v=\frac{(126)(2.5)(2000)}{(4)(30)(0.875)(41.5)}=145 \text{ lb. per square inch}$$

which is high but will be considered satisfactory here in order not to confuse the text by changing the footing depth. A value of 120 lb. per square inch is permissible for punching shear.

Where, on account of soil conditions, a greater depth is needed below the basement floor than that required for the footing, the pier or column between the top of the footing and the basement floor should be flared in a similar manner (but inverted) to the flare in columns just below flat-slab floors, as by so doing the bending moment and shear on the footing may be decreased. This should be clear from the discussion on flat-slab floors.

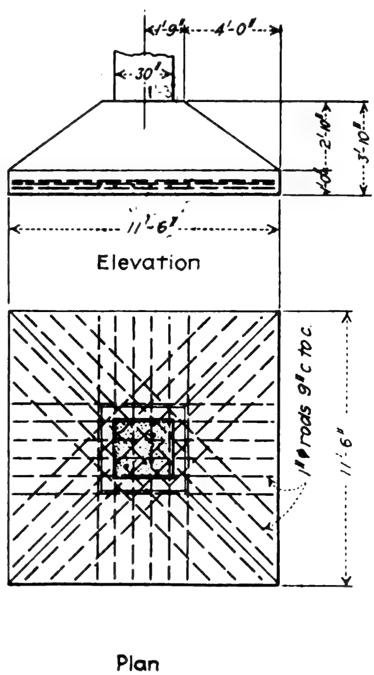


FIG. 168.

Tests have shown (Art. 18) that it makes little or no difference whether rods in a footing are placed in two or in four directions. If employed in two directions, the two sets of rods should be placed at right angles to each other and should lie parallel with the sides of the footing. Each set should be distributed over a distance somewhat larger than the diameter of the base of column. Extra rods will usually be needed by this method in each direction between the end rods of each set and the sides of the footing. The spacing of these rods may very well be made double that which is employed directly below the column.

The complete design is shown in Fig. 168.

A single symmetrical column footing may be considered as a flat slab loaded by the uniformly-distributed upward pressure of the soil and fixed rigidly to the column. The formulas given in Art. 18 for a circular plate may be applied to a square footing without appreciable error. The radii to be used in the formulas are the averages of the radii of the circumscribed and inscribed circles. In the problem we are considering

$$r_1=\frac{11.5+(1.41)(11.5)}{4}=6.93 \text{ ft.}$$

$$r_0=\frac{2.5+1.41(2.5)}{4}=1.51 \text{ ft.} \qquad \frac{r_1}{r_0}=4.6$$

From the table of Art. 18,

$$R_1=12.61$$

Then

$M=(2.5)(2000)(1.51)^2(12.61)=144{,}000$ in.-lb. per inch of circumference.

Total moment around the column base

$$M=(144{,}000)(2)(3.142)(1.51)(12)=16{,}400{,}000 \text{ in.-lb.}$$

This total bending moment by the *flat-slab* method is only about four-fifths of that given by the *cantilever* method.

The remainder of the computations by the flat-slab method will not be worked out, since they would be similar in every respect to the computations previously given.

PROBLEM

11. Using 1-in. round deformed rods, design a single square footing to sustain the load determined in Problem 10. Allowable pressure on the soil is 2 tons per square foot. Employ the working stresses recommended by the Joint Committee for a 2000-lb. concrete and medium steel. Design completely by the cantilever method and compare the bending moment thus determined with the bending moment from the flat-slab method.

Design of Footing for Wall Column

Design a single rectangular footing to support a wall column carrying a load of 400,000 lb. Consider the column rectangular in shape with dimensions 24 in. by 38 in. A 1:1½:3 concrete with

medium steel will be employed and the safe bearing capacity of the soil will be taken at $2\frac{1}{2}$ tons per square foot.

The dead weight of the footing will be assumed at 25,000 lb., making a total load on the soil of 425,000 lb.

$$\frac{425{,}000}{(2.5)(2000)} = 85 \text{ sq. ft.}$$

We shall spread the base of the footing to cover a rectangular area of 9 ft. 8 in. by 8 ft. 6 in.

Since this footing design will be used later on in the design of a three-story building, it might be well to describe here the reinforcement in the column. A $1:1\frac{1}{2}:3$ concrete will permit a compressive strength in the concrete of 560 lb. per square inch. Allowing $1\frac{1}{2}$ in. on all sides for fire protection, a plain-concrete column of the same dimensions would carry a total load of

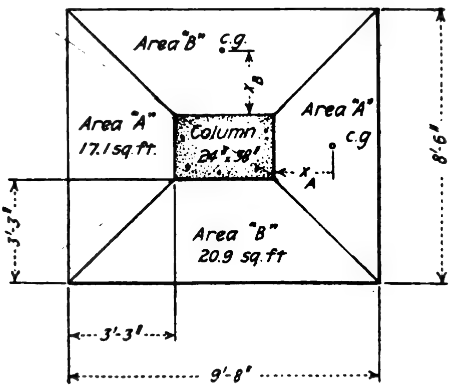


FIG. 169.

412,000 lb. (If there is no possibility of the erection of an adjacent building, a $1\frac{1}{2}$-in. protective covering need not be provided on the outside of the column.) Thus, no vertical steel is theoretically required. The three-story building to be designed later will cover considerable ground area and will be square in shape. Thus wind stresses will not be considered. It is possible to compute the stress due to eccentric loading (as will be explained in Chapter XIV), but the computations are long and, in order not to make this treatment too complex, the assumption will be made that eight 1-in. round rods will give ample provision in this respect. Hoops, $\frac{3}{8}$-in. rounds, will be

16

placed 12 in. apart the entire height of the column. The vertical rods will rest upon steel plates 3 in. square, which will distribute the compression from the steel to the concrete. Rods will be made to lap above each floor.

Let the areas and distances out from the face of the column to the centers of gravity of these areas be as represented in Fig. 169. Also let it be assumed as in the preceding design that the weight of footing beyond the edge of the column need not be considered as opposing the upward thrust of the earth. Then we have the following (see preceding design):

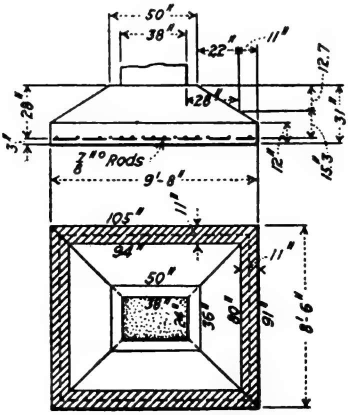


Fig. 170.

$$x_A = \frac{\frac{(2)(3.25)}{2} + \frac{2}{3}(3.25)^2}{2+3.25} = 1.96 \text{ ft.}$$

$$x_B = \frac{\frac{(3.17)(3.25)}{2} + \frac{2}{3}(3.25)^2}{3.17+3.25} = 1.90 \text{ ft.}$$

$$M_A = (17.1)(2.5)(2000)(1.96)(12) = 2{,}010{,}000 \text{ in.-lb.}$$
$$M_B = (20.9)(2.5)(2000)(1.90)(12) = 2{,}380{,}000 \text{ in.-lb.}$$

$$d_A = \sqrt{\frac{2{,}010{,}000}{(107.4)(24)}} = 28.0 \text{ in.}$$

$$d_B = \sqrt{\frac{2{,}380{,}000}{(107.4)(38)}} = 24.2 \text{ in.}$$

The depth of the footing is controlled by the moment of the *A* areas, and the rods in the longitudinal direction will be placed 28 in. down from the top of the footing and 3 in. from the bottom—making a total depth of footing of 31 in. The rods in a crosswise direction will be placed on top of the longitudinal rods. (See Fig. 170.)

Longitudinal direction:

$$a_s = (0.0077)(24)(28) = 5.2 \text{ sq. in.}$$

Nine $\frac{7}{8}$-in. round rods O.K.

$$V = \frac{(91)(11)(2.5)(2000)}{144} = 34,800 \text{ lb.}$$

$$v = \frac{34,800}{(80)(0.875)(15.3)} = 33 \text{ lb. per square inch.}$$

$$u = \frac{(17.1)(2.5)(2000)}{(9)(2.75)(0.875)(28)} = 141 \text{ lb. per square inch.}$$

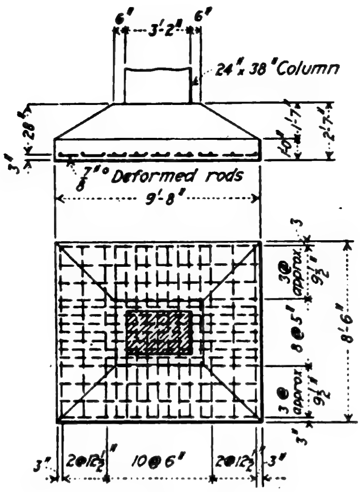


Fig. 171.

Crosswise direction:

$$K = \frac{2,380,000}{(38)(27)^2} = 86.0$$

$$p = 0.0061$$

$$a_s = (0.0061)(38)(27) = 6.3 \text{ sq. in.}$$

Eleven $\frac{7}{8}$-in. round rods O.K.

$$V=\frac{(105)(11)(2.5)(2000)}{144}=40,000 \text{ lb.}$$

$$v=\frac{40,000}{(94)(0.875)(14.3)}=34 \text{ lb. per square inch.}$$

$$u=\frac{(20.9)(2.5)(2000)}{(11)(2.75)(0.875)(27)}=147 \text{ lb. per square inch.}$$

A good type of deformed rod will be employed. Punching shear is not much more than the allowable. The complete design is shown in Fig. 171.

PROBLEM

12. Design a wall column footing to sustain a rectangular column 24 in. by 36 in., carrying 400,000 lb. Take the allowable pressure on the soil at 2 tons per square foot. Use 1-in. round deformed rods and employ the working stresses recommended by the Joint Committee for a 2000-lb. concrete with medium steel reinforcement.

48. Combined Column Footings.—It is sometimes necessary to support a column on, or very near, the edge of a property line in order not to encroach upon adjacent property. In such a case a single symmetrical footing cannot be used. The nearest interior column is usually selected and a combined footing constructed under both columns. Sometimes a combined footing will include more than two columns. The following problem in design will show the general manner of determining the shape of such footings, also the necessary depth and amount of steel.

DESIGN OF COMBINED FOOTING

Design a combined footing for columns 1 and 2, Fig. 172. Column 1 is 20 in. square and sustains a load of 280,000 lb. Column 2 is 24 in. square and sustains a load of 380,000 lb. Distance between centers of columns is 14 ft. Consider that a soil pressure of 3 tons per square foot is somewhat below the safe bearing capacity, so that the dead weight of footing need not be considered. Assume a 2000-lb. concrete with medium steel reinforcement.

The design of combined footings consists in constructing a base, the center of gravity of which coincides with the center of gravity of the two unequal loads. In addition, the base must

have sufficient area so that the allowable pressure on the soil will not be exceeded.

Load on column 1 = 280,000 lb.
Load on column 2 = 380,000 lb.
Total = 660,000 lb.

$$\frac{660{,}000}{6000} = 110 \text{ sq. ft.}$$

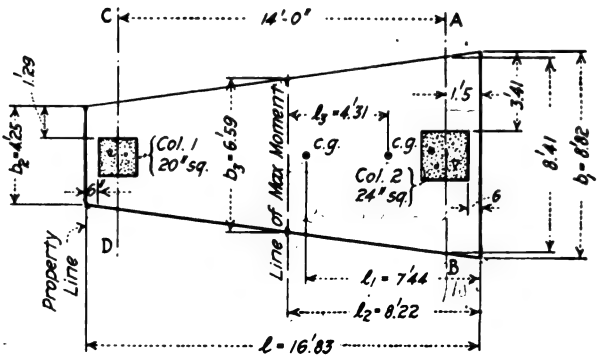


FIG. 172.

The lengths of the parallel sides are unknown and two equations will be needed to solve. First equation may be obtained from the formula that the area of a trapezoid equals the average of the sum of the parallel sides multiplied by its length. The second equation may be found from the principle stated above—that the center of gravity of the trapezoid must coincide with the center of gravity of the combined column loading. This must be so in order that the pressure on the soil, and the consequent settling (if any), may be uniform—also to prevent dangerous transverse stresses in the columns.

$$\text{Area of footing} = \frac{b_1+b_2}{2}(16.83) = 110$$

$$b_1+b_2 = 13.07 \qquad (1)$$

$$l_1 - 1.5 = \frac{280{,}000}{660{,}000}(14) = 5.94$$

or

$$l_1 = 7.44 \text{ ft.}$$

Using the common equation for the center of gravity in a trapezoid (Cambria Handbook)

$$l_1 = 7.44 = \frac{16.83}{3} \cdot \frac{b_1+2b_2}{b_1+b_2} \qquad (2)$$

Solving equations (1) and (2) for b_1 and b_2

$$b_1=8.82 \text{ ft., and } b_2=4.25 \text{ ft.}$$

To find the necessary depth of footing and amount of steel required, we must find the section of maximum moment (which is the section of zero shear) and then determine the center of gravity of the area to one side of this section as an aid in finding the value of this maximum moment.

$$\left[8.82\, l_2-\frac{8.82-4.25}{(2)\,(16.83)}\, l^2_2\right]6000=380,000$$

$$l_2-0.0154\, l^2_2=7.18$$

$$l_2=8.22$$

And by trial,

$$b_1-b_2=8.82-4.25=4.57$$

$$\frac{b_3-b_2}{4.57}=\frac{16.83-8.22}{16.83}$$

$$b_3-b_2=2.34$$

$$b_3=2.34+4.25=6.59$$

$$l_3=\frac{l_2}{3}\cdot\frac{b_3+2b_1}{b_3+b_1}=\frac{8.22}{3}\cdot\frac{6.59+2(8.82)}{6.59+8.82}=4.31$$

We can now compute the value of the maximum moment.

$$M=380,000\ (8.22-1.5-4.31)=915,800 \text{ ft.-lb.}$$

or $$M=(915,800)(12)=10,989,600 \text{ in.-lb.}$$

The moment for 1 in. of width along the line of maximum moment, $M=\dfrac{10,989,600}{(6.59)(12)}=139,000$ in.-lb. The footing must be considered as an inverted beam at this section, and the required depth and the area of the steel may be computed by the usual methods.

If the moment had been taken about a line through the center of gravity of the entire trapezoid, the result would be 133,000 in.-lb., or an error of only about 2.2 per cent. This approximate solution is often used in practice as the error is small.

From Table 2, Volume I, $K=107.4$ and $p=0.0077$ for the stresses as recommended by the Joint Committee.

$$d=\sqrt{\frac{139,000}{107.4}}=36.0 \text{ in.}$$

$a_s=(0.0077)(12)(6.59)(36)=21.9$ sq. in. for the whole width. Referring to Table 1, eighteen 1¼-in. round rods will be needed.

The bond stress along the main tension rods must now be in-

vestigated. The maximum shear comes from column 2 and equals 380,000 lb.

Assuming that deformed rods are used, then the number of rods required for bond will be (assuming $u=150$)

$$u=\frac{V}{\Sigma ojd'}\text{ or number of rods }=\frac{380{,}000}{(3.93)(0.874)(36)(150)}=21$$

Thus bond controls the number of horizontal rods required and 21 rods will be employed. (See Fig. 173.)

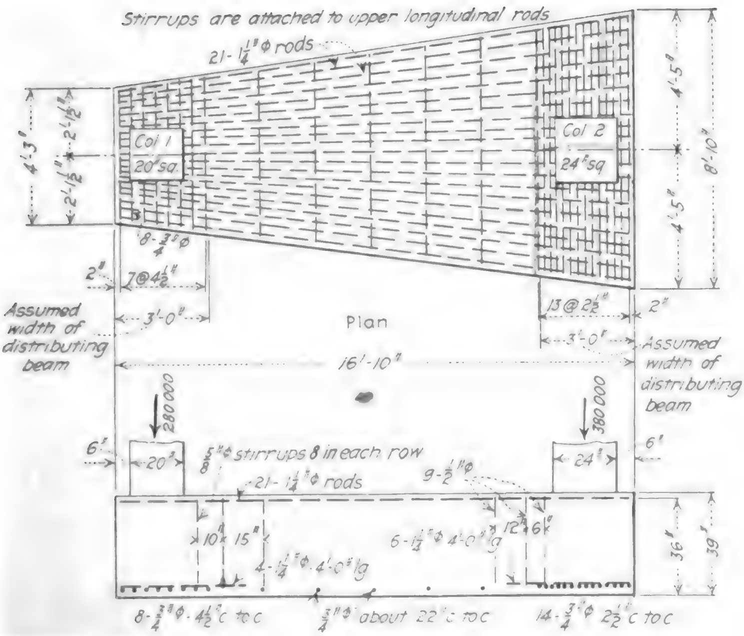


FIG. 173.

Transverse reinforcement should be provided to prevent bending of the projections of the footing. Considering column 2, the width of distributing beam will be taken at 3 ft. Now the loading for which this cantilever should be designed is equal to one-half of the column loading multiplied by $\frac{8.82-2}{8.82}$. The moment arm equals one-half of the length of the projection, and

$$M=\left(\frac{380{,}000}{2}\right)\left(\frac{8.82-2}{8.82}\right)\left(\frac{3.41}{2}\right)(12)=3{,}010{,}000\text{ in.-lb.}$$

$$d=\sqrt{\frac{3{,}010{,}000}{(107.4)(3.0)(12)}}=27.9$$

The depth is smaller than the depth of the whole slab and, since a greater depth is used, the percentage of steel is found as follows:

$$K=\frac{M}{bd^2}=\frac{3,010,000}{(36)(36)(36)}=64.6$$

From Table 3,

$$p=0.0045$$

Then,

$$a_s=(0.0045)(36)(36.0)=5.84 \text{ sq. in.}$$

Referring to Table 13, fourteen ¾-in round rods will be needed.

In the same manner, the distributing reinforcement for column 1 is determined.

$$M=\left(\frac{280,000}{2}\right)\left(\frac{4.25-1.67}{4.25}\right)\left(\frac{1.29}{2}\right)(12)=658,000 \text{ in.-lb.}$$

$$d=\sqrt{\frac{658,000}{(107.4)(36)}}=1.31 \text{ in.}$$

Since the depth of the whole slab must be used, the necessary percentage of steel is found as above.

$$K=\frac{M}{bd^2}=\frac{658,000}{(36)(36)^2}=14.2$$

Only about 0.1 per cent of steel is required and three ¾-in. round rods will suffice.

It sometimes happens that the required depth of the distributing beam may be larger than the depth of the whole slab. In such cases the footing may have an increased thickness under the column or else steel may be introduced at the top and bottom. Double reinforcement, however, should not be employed when excavation can readily be made.

The bond stress along the rods of the distributing beams must now be investigated. Considering the beam under column 2, the maximum shear is.

$$\left(\frac{380,000}{2}\right)\left(\frac{8.82-2.0}{8.82}\right)=147,000 \text{ lb.}$$

Assuming that deformed rods are used, then the number of rods required for bond will be ($u=150$)

$$u=\frac{V}{\Sigma ojd}, \text{or number of rods} =\frac{147,000}{(2.36)(0.898)(36)(150)}=13.$$

Thus the number determined for moment controls. In a similar manner, 8 rods are needed in the distributing beam under column 1 but in this latter case the number is greater than that required for moment.

Fig. 173 shows the spacing of the transverse reinforcement. Steel should be introduced between the end distributing beams about 24 in. center to center in order to make the footing more rigid and more capable of directly transmitting the loads.

Certain assumptions must be made in order to determine the amount of reinforcement required for diagonal tension, but the assumptions which will be made will be at least on the safe side. Referring to Fig. 174 it will be assumed that the shear at the section *mn* is due to the load from column 2 minus the pressure

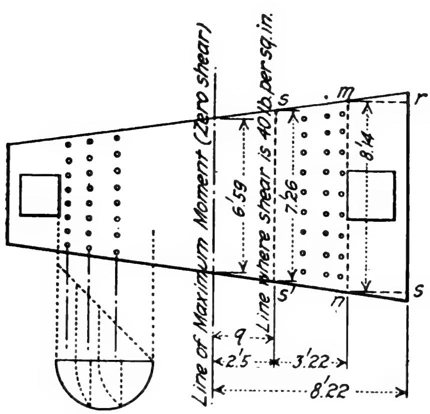


Fig. 174.

on the soil over the area *mrsn*. Web reinforcement is not needed in the distributing beams, as each distributing beam and column has a similar load to that of a single footing, and for such footings we know that the intensity of shearing stress, as measuring diagonal tension, may be computed on a section at a distance out from the face of column equal to the depth of the footing to the steel. In a longitudinal direction, however, shear should be computed on a section close to the support, as *mn*, Fig. 174, since in this direction the tension steel is at the top of the footing and cracks may open up in the web as in simple beams.

In order to determine the number of stirrups required in a longitudinal direction at column 2, it will be necessary to find

the section where the shear is 40 lb. per square inch. This section may be determined approximately as follows (Fig. 174):

$$(6.59)(q)6000) = (36)(6.59)(12)(40)(\tfrac{7}{8})$$
$$q = 2.5 \text{ ft.}$$

This value is sufficiently accurate for all practical purposes. If desired, the distance q may be determined by trial and by scaling from an accurate drawing of the trapezoid. The distance $mn = 8.14$, determined directly by scaling. Also, in the same manner, $ss' = 7.26$. At mn the unit shear is

$$\frac{380{,}000 - (8.14)(2.5)(6000)}{(8.14)(36)(12)(\tfrac{7}{8})} = 84 \text{ lb. per square inch.}$$

The total shear to be taken by vertical stirrups is

$$\left(\frac{84-40}{84}\right)\left(\frac{84}{2}\right)\left(\frac{8.14+7.26}{2}\right)(3.22)(144) = 79{,}000 \text{ lb.}$$

Using $\tfrac{1}{2}$-in. round rods, the number of single stirrups required is

$$\frac{79{,}000}{(16{,}000)(0.196)} = 25$$

The spacing is given in Figs. 173 and 174 (27 stirrups shown). In a similar manner the number of stirrups may be determined at column 1.

The loads beyond the centers of the columns in a longitudinal direction cause negative moment at the supports; that is, the load to the left of column 1 and to the right of column 2 cause tensions in the bottom of the footing at the sections AB and CD respectively (Fig. 172). This tension, however, does not exceed the tensile strength of the concrete. At column 1, using the flexure formula given in Art. 29, Volume I

$$f = \frac{My}{I} = \frac{(8.82)(1.5)(6000)(9)(18)(12)}{(8.41)(12)(36)^3} = 33 \text{ lb. per square inch.}$$

A few rods 4 ft. long will be placed at the bottom of footing across the sections AB and CD in order to provide for any possible defects in the bed of the foundation.

There is ample provision for punching shear around the columns.

Whenever it becomes necessary to employ a combined footing which will include two columns carrying practically the same load, then an inverted T-beam will answer and the method of

design shown in the illustrative problem following Art. 62, Volume I, should be used.

PROBLEM

13. In Fig. 173 consider a load of 200,000 lb. on column 1 and a load of 300,000 lb. on column 2. Take a distance of 15 ft. between the centers of columns. Design the combined footing considering an allowable pressure on the soil of 2 tons per square foot. Use the working stresses as recommended by the Joint Committee for a 2000-lb. concrete and medium steel.

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49. Cantilever Construction.—The cantilever type of construction may be employed in place of a combined footing under the usual conditions; that is, when encroachment upon adjacent property must be avoided and when, at the same time, it becomes necessary to make use of the land close to the property line. In cantilever construction, the wall-column footing and the footing of the nearest interior column are connected by a beam or strap, and this strap is extended so as to support the wall column. To save excavation, the top of the strap is usually placed at the same level as the tops of the footings.

Design of Footings in Cantilever Construction

Let us assume the size of columns and column loading as shown in Fig. 175. The distance from the center of interior column to the property line will be made 12 ft. and the outer edge of the wall column will be placed on the property line. In addition to the above, we shall consider that piles are necessary and that it is not feasible to drive such piles closer than 2 ft. apart in any direction. The safe bearing capacity of each pile will be taken at 10 short tons. The working stresses as recommended by the Joint Committe for a 2000-lb. concrete and medium steel will be adopted.

Approximate figuring shows that about 12 piles will be needed under the wall footing, and 18 piles under the interior footing; that is, if the center of the wall footing is placed about as shown in Fig. 175. Knowing the number of piles required, the shape of the footings may be easily determined. The arrangement shown for the piles insures a spacing of at least 2 ft. in every direction.

On account of cantilever action, the uplift on the interior column may be due to the combined live and dead load of the

wall column, but, since the live load may not always be present, the uplift from the dead load only should be considered. Dis-

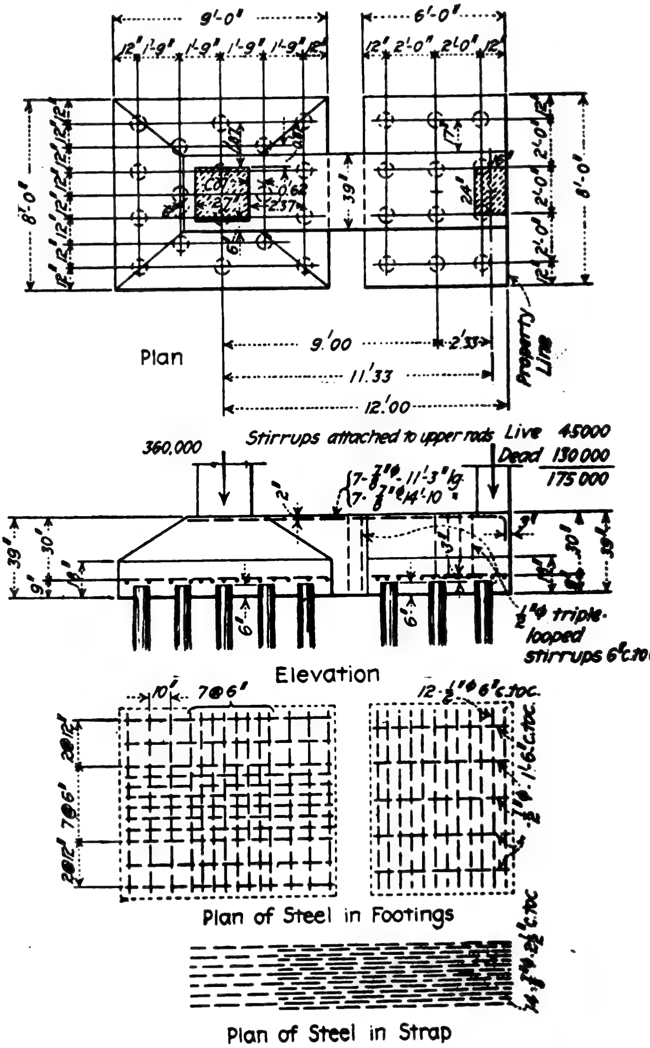

FIG. 175.

regarding the strap weight to the right of the center of wall footing, this uplift is

$$\frac{(130{,}000)(2.33)-(1200)(9)(4.5)}{9.00}=28{,}000 \text{ lb.}$$

The strap will now be designed for the total live and dead load on the exterior column since this loading gives maximum conditions. The total load on the wall footing is

$$\frac{(175,000)(11.33)+(1200)(12)(6)}{9.00}=230,700 \text{ lb.}$$

The uplift on the interior column may be found in a similar manner to that for dead load only, or more accurately (taking moments about the outer edge of the wall column)

$$\frac{(230,700)(3.00)-(175,000)(0.67)-(1200)(12)(6)}{12.00}=40,700 \text{ lb.}$$

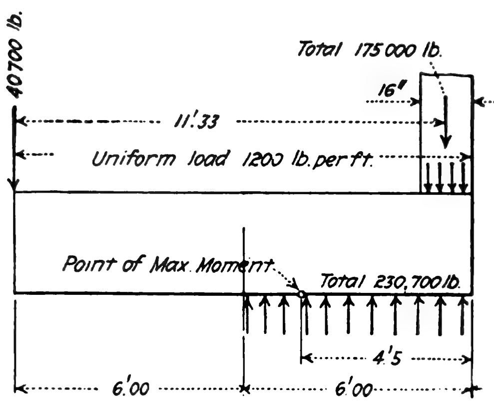

FIG. 176.

The loads on the strap are shown in Fig. 176. The maximum moment occurs where the shear is zero, or accurately enough,

$$\frac{(175,000)(6)}{230,700}=4.5 \text{ ft.}$$

from the outside edge of wall column.

$$M=(175,000)(3.83)-\left(\frac{230,700}{6}\right)(4.5)(2.25)=281,000 \text{ ft.-lb.}$$

$$=3,372,000 \text{ in.-lb.}$$

Then, assuming $b=39$ in.,

$$d=\sqrt{\frac{3,372,000}{(107.4)(39)}}=28\tfrac{1}{2} \text{ in.}$$

The shear in the strap at the inside edge of wall column should not exceed 120 lb. per square inch with an effective system of web reinforcement. This is due to the fact that tension exists in the

upper portion of the strap; in orther words, the inward spread of the wall-column load will not aid to any appreciable extent in reducing the tendency to fail through diagonal tension. For the dimensions determined for moment

$$v=\frac{(175{,}000)-\left(\frac{230{,}700}{6}\right)(1.33)}{(39)(0.875)(28.5)}$$

$$=\frac{123{,}900}{(39)(0.875)(28.5)}=127 \text{ lb. per square inch.}$$

The strap will be deepened so that $d=30$ in. and total depth $=32$ in. Stirrups will be needed from the inner edge of the exterior column to a point 3.5 ft. from the property line where the shear becomes 40 lb. per square inch. Triple-looped ½-in. round stirrups will be employed and these will be spaced 6 in. on centers, or

$$s=\frac{3}{2}\cdot\frac{a_sf_sjd}{V}=\frac{3}{2}\cdot\frac{(6)(0.196)(16{,}000)(0.875)(30)}{123{,}900}=6 \text{ in.}$$

The unit shear at the inner edge of wall footing is

$$\frac{(40{,}700)+(1200)(6)}{(39)(0.875)(30)}=47 \text{ lb. per square inch.}$$

Two triple-looped ½-in. round stirrups placed just to the left of this point will make the design satisfactory.

The number of horizontal rods will now be determined.

$$K=\frac{3{,}372{,}000}{(39)(30)^2}=96$$

$$p=0.0069$$

$$a_s=(0.0069)(39)(30)=8.1 \text{ sq. in.}$$

Fourteen ⅞-in. round deformed rods O.K.

$$u=\frac{123{,}900}{(14)(2.75)(0.879)(30)}=122 \text{ lb. per square inch.}$$

The horizontal tension rods should have a sufficient length for straight bond on each side of the section of maximum moment. In the problem at hand, if deformed bars are assumed having a bond strength of at least 150 lb. per square inch, then 27 diameters or 24 in. is sufficient. It is advisable, however, in a construction of this kind to bend the ends of the rods as shown. Alternate rods may stop off at 10 ft. from the property line, which is about 5 in. beyond the point where they are no longer needed in tension.

The interior footing may be designed in the same manner as

explained in Art. 47, except that in this problem we have the moment from concentrated loads instead of from uniform loads.

$$M = (80{,}000)(2.37)(2)+(20{,}000)(0.62)(2)+(20{,}000)(1.87)(2)$$
$$+(40{,}000)(0.87)(2)$$
$$= 548{,}400 \text{ ft.-lb., or } 6{,}581{,}000 \text{ in.-lb.}$$

The depth to steel will be made 30 in. in order to provide properly for the strap. Web reinforcement is not needed.

$$K=\frac{6{,}581{,}000}{(27)(4)(30)^2}=68$$
$$p=0.0048$$
$$a_s=(0.0048)(27)(4)(30)=15.6 \text{ sq. in.}$$

The steel will be placed in two directions. Thus, for moment, 3.9 sq. in. or seven $\frac{7}{8}$-in. round rods will be needed in each band. Let us now determine the number of $\frac{7}{8}$-in. round rods in each set that will be required for bond.

$$u=\frac{V}{\Sigma ojd}\text{, or number of rods}=\frac{\frac{(16)(20{,}000)}{4}}{(150)(2.75)(0.895)(30)}=8$$

This number controls.

The weight of footing, as designed, deducting the weight of strap already considered, is approximately 22,000 lb. This gives a total pressure on the piles of $360{,}000+22{,}000-28{,}000=354{,}000$ lb. Thus the number of piles was correctly chosen. In this design there is no danger of failure by direct shear above the tops of the piles as for this kind of shear about one-half the compressive strength of the concrete may be allowed. There is also no danger from punching shear around the base of the column.

The footing under the wall column acts as a simple cantilever. The moment on each edge of strap

$$M=(60{,}000)(17)=1{,}020{,}000 \text{ in.-lb.}$$

As in the interior footing, the depth to steel will be made 30 in.

$$K=\frac{1{,}020{,}000}{(72)(30)^2}=16$$
$$p=0.0011$$
$$a_s=(0.0011)(72)(30)=2.4 \text{ sq. in.}$$

Twelve $\frac{1}{2}$-in. round deformed rods.

$$u=\frac{60{,}000}{(12)(1.57)(0.93)(30)}=115 \text{ lb. per square inch.}$$

A few cross-rods will also be employed to better distribute the load.

The weight of wall footing deducting the weight of strap is approximately 10,000 lb., giving a total pressure on the piles of 230,700+10,000=240,700 lb., or almost exactly 10 tons per pile, as planned.

It is generally considered good practice to lay the concrete directly upon the heads of the piles. The ground is usually excavated around the piles, the depth depending upon soil conditions, and then a layer of broken stone is spread and rammed before the concrete is laid. The supporting power of the soil between the piles is thus utilized.

PROBLEM

14. Design a cantilever type of construction for the conditions of Problem 13 using piles with an allowable load of 20,000 lb. per pile.

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50. Raft Foundations.—The raft foundation in building construction may be considered as an extension of the single or combined footing until the foundation covers at least a considerable, if not the entire, area of the building. Its use is limited to sites where the allowable pressure on the soil is very small or where the building is supported by piles sustained by friction.

Raft foundations may be divided into the following three general classes:

1. Flat slabs of plain or reinforced concrete.
2. Beams or girders with a slab underneath.
3. Beams or girders with a slab on top.

A flat-slab foundation may be designed in the same manner as a flat-slab floor using either of the methods described in Art. 18. The foundation is considered as an inverted flat slab loaded by the reaction of the ground and supported by the columns. The column base should be made large enough to prevent excessive moments and shears in the concrete.

Fig. 177 shows a design of a raft foundation of the second class. The action of forces is exactly as in an inverted floor and calls for similar treatment in designing. As compared with Class 3, Class 2 permits a T-beam design but, on the other hand, requires an extra fill and separate floor surface in the basement.

51. Examples of Column Footings.—Fig. 178 shows a design of a single footing for the typical interior columns of a factory building for the Bradley Knitting Co., Delavan, Wis. The design of the top of footing should be noted.

Fig. 179 is an example of a combined footing employed in an automobile depot in Boston.

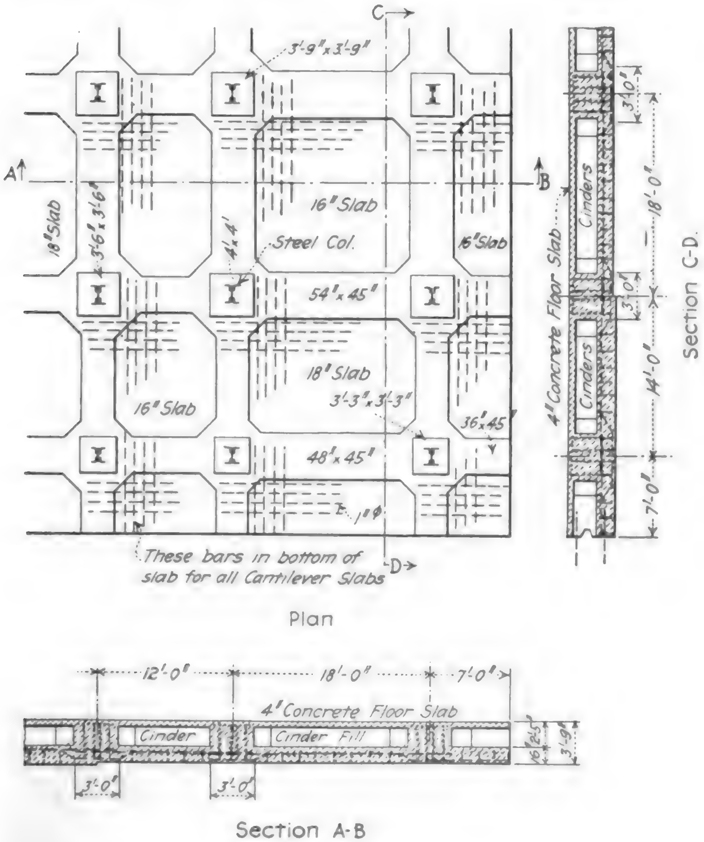


FIG. 177.

Fig. 180 is a special foundation of novel design used in the Garage Building of the Decanville Automobile Co., New York City.

Figs. 181A to 181E inclusive show an interesting foundation arrangement employed in a storehouse for the Winchester Repeating Arms Co., New Haven, Conn. The combined

footings and the cantilever footings for the bearing walls should be noted. The building was designed for a pressure of 6000 lb. per square foot on the earth under the foundations. The entire wall and footing arrangement will be considered more in detail in Art. 53.

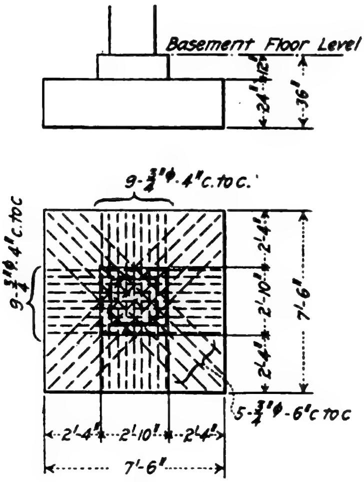
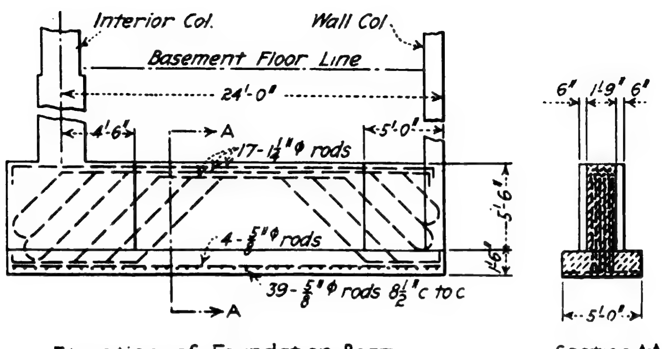


FIG. 178.



Elevation of Foundation Beam

Section AA

FIG. 179.

52. Concrete Piles.—Reinforced-concrete piles are usually employed where wooden piles would be subject to decay or to destruction by the action of marine worms. They may be used generally under the same conditions as wooden piles, but are especially advantageous for foundations on land where the per-

manent ground water is at a considerable depth. Wooden piles must be cut off under water as, when subjected to an atmosphere which is alternately wet and dry, they will decay. This is unnecessary with concrete piles, and foundations under such conditions need not start so low as would be the case if

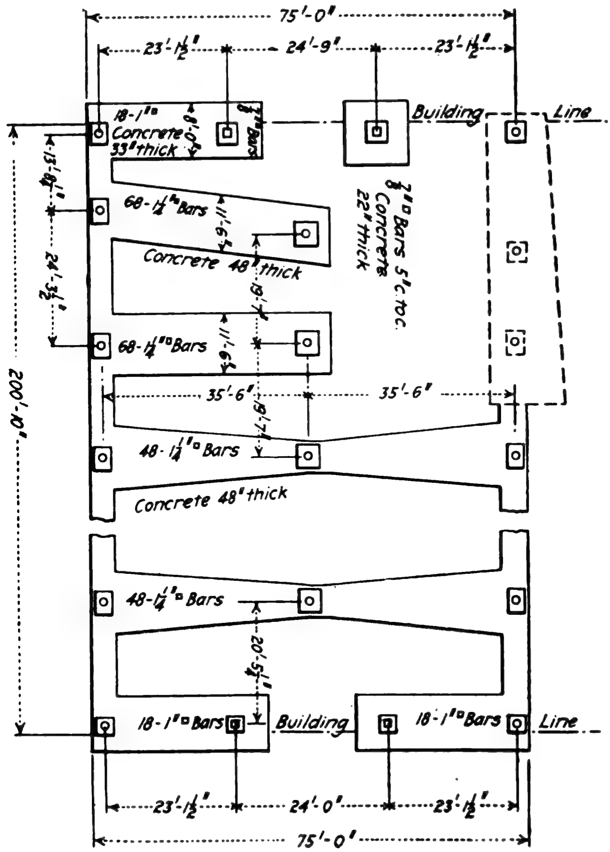


Fig. 180.

timber piles were employed. Concrete piles cost somewhat more than timber piles, but under many circumstances the saving in foundation material and in excavation will more than offset the extra cost of the concrete pile.

Reinforced-concrete sheet piling is sometimes used when it is desirable to leave it in place as a part of the permanent structure.

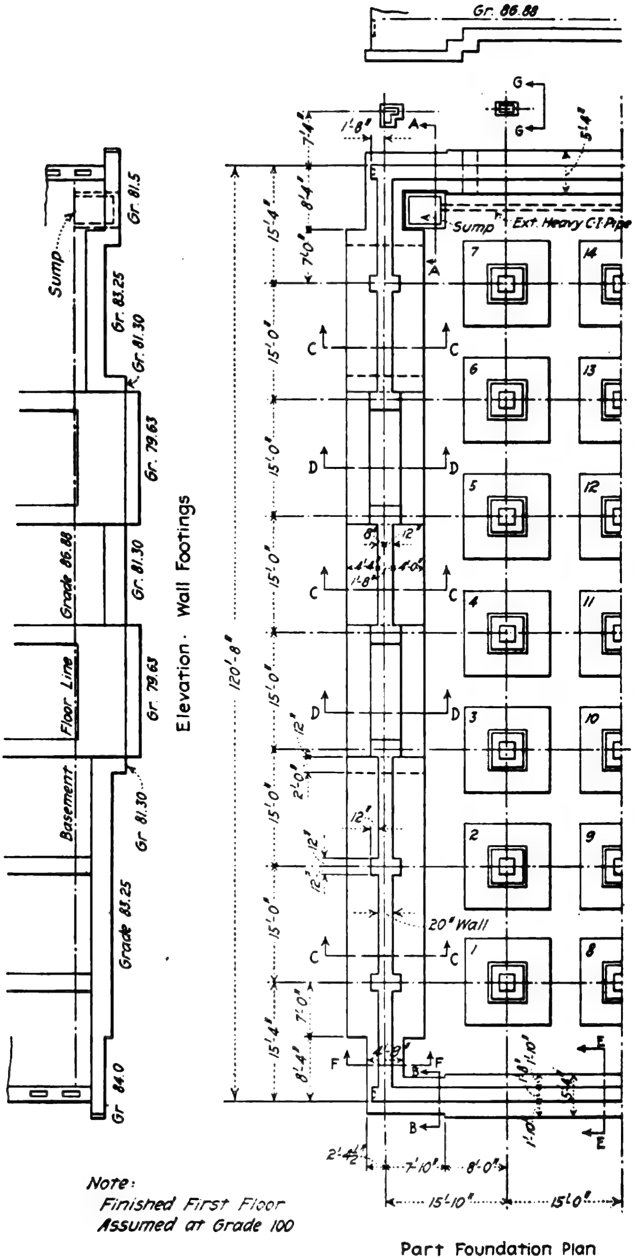


FIG. 181A.—Winchester Repeating Arms Co.'s building, New Haven, Conn.

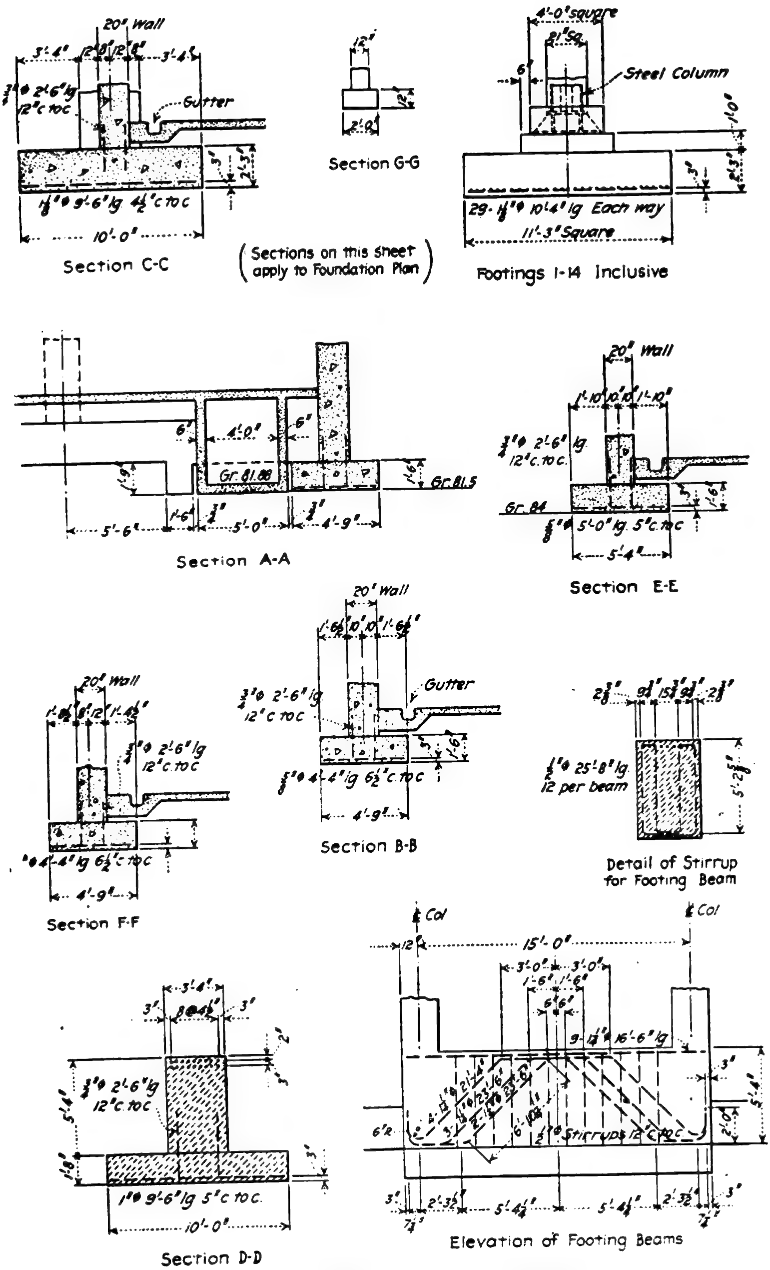


FIG. 181B.—Winchester Repeating Arms Co.'s building, New Haven, Conn.

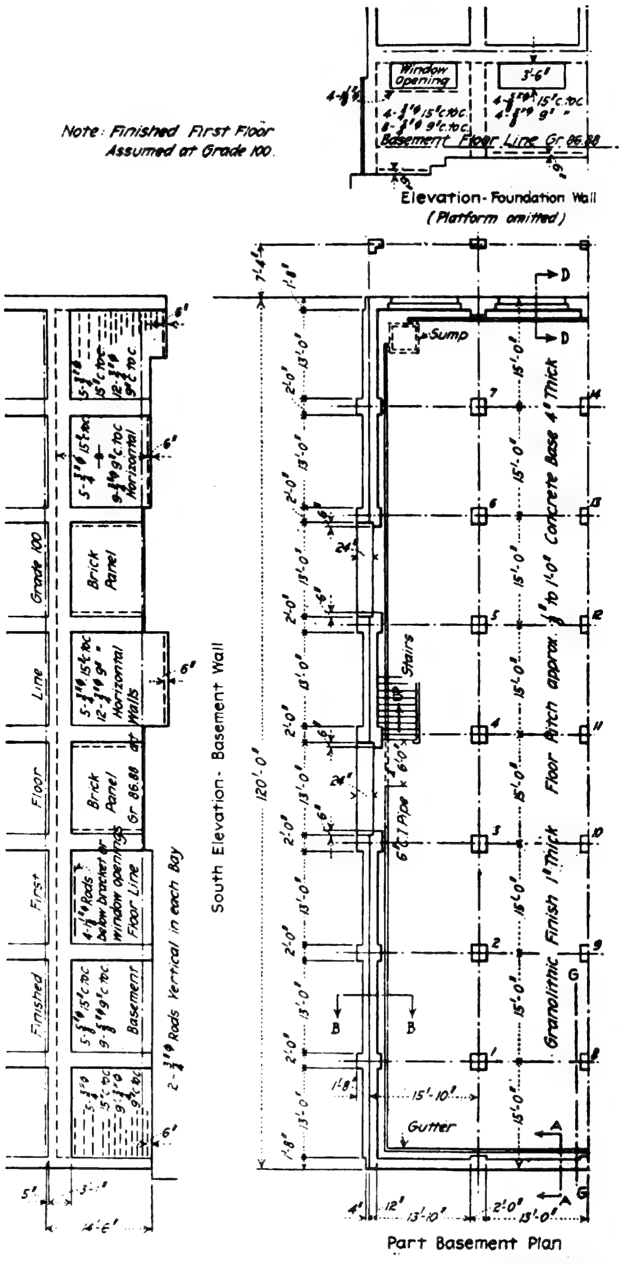


FIG. 181C.—Winchester Repeating Arms Co.'s building, New Haven, Conn.

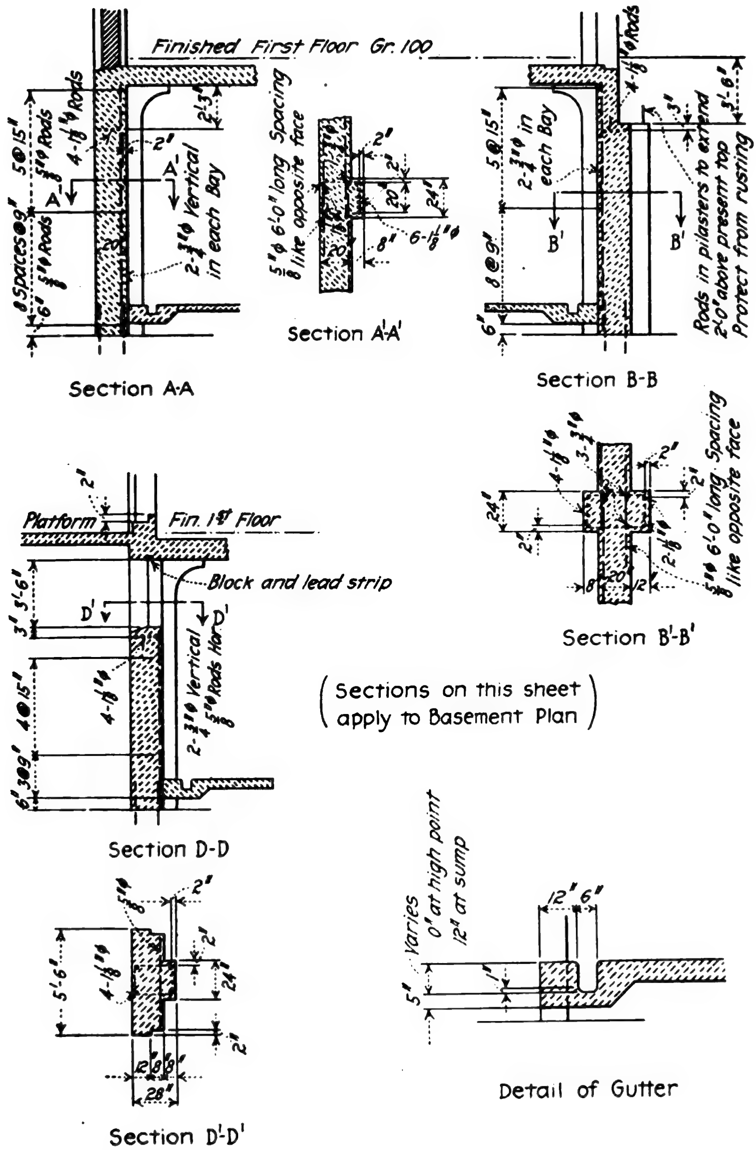


FIG. 181D.—Winchester Repeating Arms Co.'s building, New Haven, Conn.

In this type of piling, grooves are left in adjacent edges of the piles, and these grooves are filled with grout after the piles are driven.

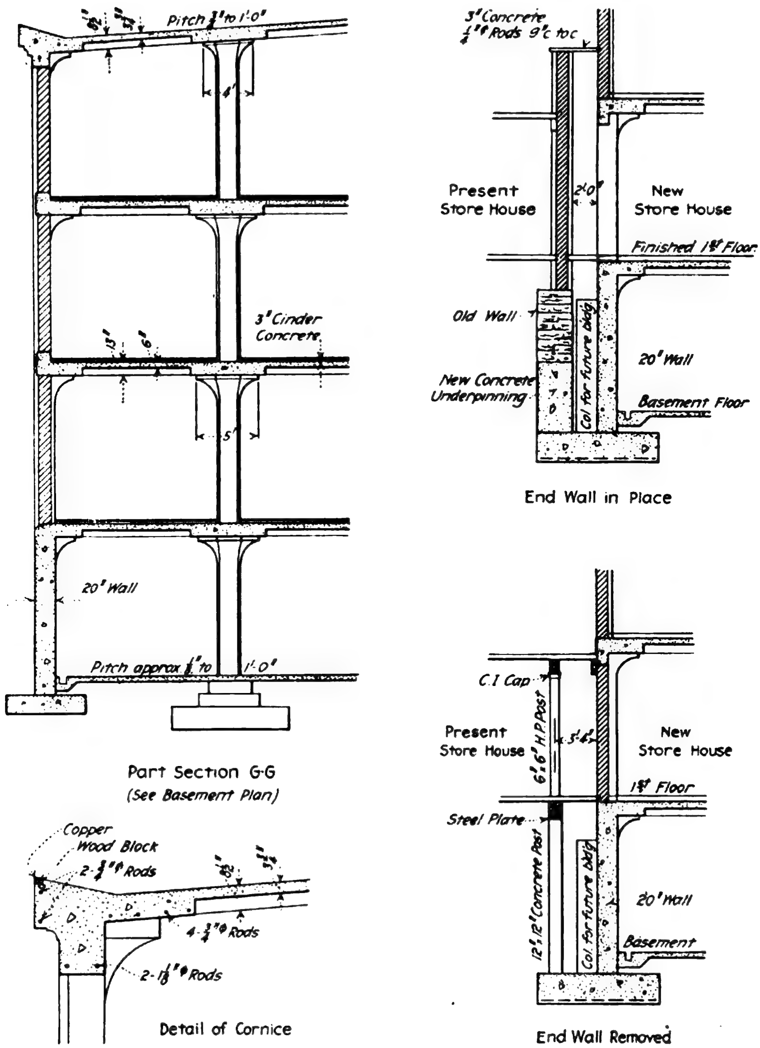


FIG. 181E.—Winchester Repeating Arms Co.'s building, New Haven, Conn.

Reinforced-concrete piles are of two general types, those molded in place and those molded before driving.

Piles Molded in Place

The Simplex and the Raymond piles are the only forms of concrete piles molded in place which are widely used.

Simplex Pile.—In the Simplex pile a hollow cylindrical steel tube or form, 16 in. in diameter and $\frac{3}{4}$ in. thick, is driven to a suitable bearing. When the required depth is reached, the form is filled with concrete to a sufficient height, and then withdrawn. The driving point, depending on soil conditions, may be either a conical cast-iron point (Fig. 182) that is left in place, or a hinged cutting edge called an alligator point (Figs. 183 and 184) which opens as the tube is withdrawn. Fig. 185 shows the standard Simplex pile which fills the great majority of requirements and is adaptable in all ordinarily unreliable ground. The method shown is termed "Standard" as about 75 per cent of Simplex piling stands on the detachable cast-iron point. The alligator point is limited in its use to a certain class of soil—that is, one that stands well after penetration. As generally used, no reinforcement is required, but the system admits of placing any desired reinforcing members within the form. In very wet soil a permanent casing of slightly less diameter than the inside of the cylindrical form is used to prevent washing or displacement of the concrete. Sometimes a pile is molded on the surface and lowered through the form, which is then withdrawn, leaving the molded pile in position.

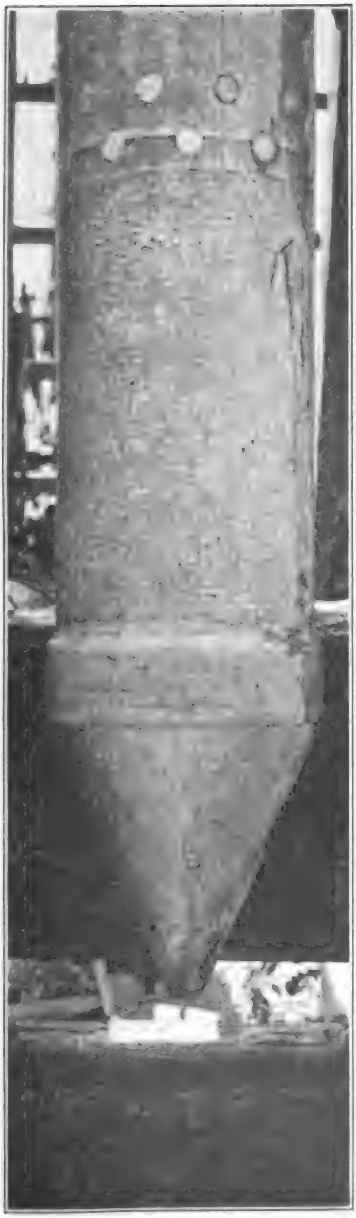
Raymond Pile.—Raymond concrete piles are made by driving a tapering sheet-steel shell to refusal by means of a collapsible steel core, withdrawing the core, and thereupon filling the shell with concrete. There is thus a shell or form for every pile and the concrete within the shell may be reinforced or not, as desired. Fig. 186 shows a partly and an entirely completed Raymond pile. The dimensions of the standard sizes of Raymond concrete piles are as follows:

20 ft. long, 20 in. at top and 6 in. at the point
25 ft. long, 20 in. at top and 8 in. at the point
30 ft. long, 20 in. at top and 8 in. at the point
35 ft. long, 18 in. at top and 8 in. at the point
40 ft. long, 18 in. at top and 8 in. at the point

Where rock or hardpan is overlaid by a stratum of soft or unstable material and a pile must be considered to act as a column, cores are used which are more nearly straight. In soft material,

Courtesy of Simplex Foundation Co.

FIG. 182.—Setting driving form on cast-iron point. Simplex concrete piles.



Courtesy of Simplex Foundation Co.

Fig. 183.—Alligator point closed in position for driving. Simplex concrete piles.

Courtesy of Simplex Foundation Co.

Fig. 184.—Alligator point open while pulling. Simplex concrete piles.

however, a tapered pile appears to have a greater bearing capacity than a straight pile of the same average diameter.

The shell of a Raymond pile is made of sheet steel of a thickness depending on the elasticity and crushing tendency of the earth. The sections are approximately 8 ft. long, telescoping into each other from 9 to 18 in. A boot made of pressed steel is placed on the end of the core or mandrel. This is designed to withstand the cutting effect of rock and other obstructions

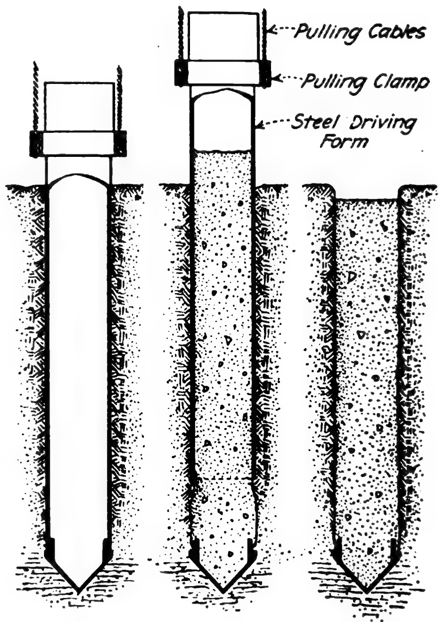


Fig. 185.—Method of constructing Simplex concrete piles.

encountered in driving. After the core is collapsed and withdrawn, the shell is inspected to see that a perfect shell has been secured, and the form is then filled with concrete.

A mechanical problem that for several years taxed the ingenuity of engineers connected with the Raymond system, was to provide a shell of sufficient strength to withstand the back pressure of ground of an elastic nature. A successful method of overcoming this difficulty is illustrated by the shell shown in cross-section in Fig. 187, termed a "spiral shell." The method consists of encasing a spiral made of No. 3 wire in a No. 24 gage shell, and to do this, special machines have been built.

Fig. 188 shows a typical Raymond pile-driving outfit. Note the core in the leads and the sections of permanent shell on the ground. These shells are placed upon the core before driving. Fig. 189 is a view of a pile driver similar to that shown in the preceding figure, but with a different size of core.

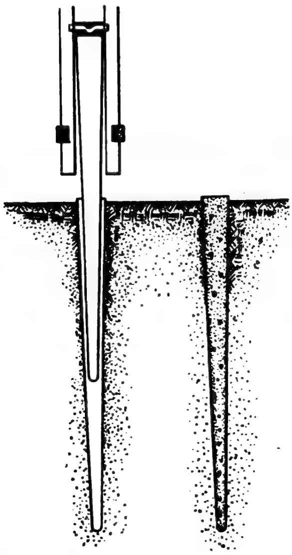


Fig. 186.—Raymond pile core collapsed and partly withdrawn. Completed Raymond pile without reinforcement.

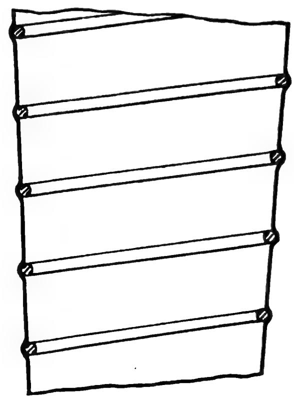
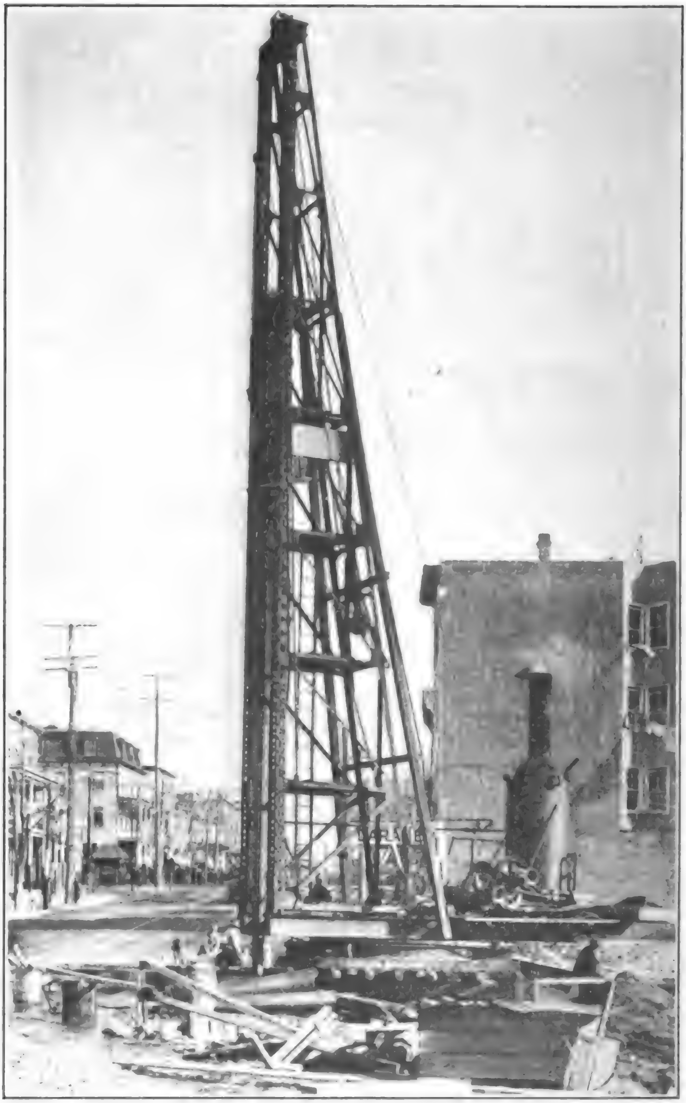


Fig. 187.

Piles Molded Before Driving

Cast piles should be reinforced to permit of handling and the reinforcement may be any of the forms used for reinforced-concrete columns. In driving, a molded pile is provided with a cast-iron point, and a driving head is generally used in which a cushion of sand, rope, or other material is placed between a driving block of wood and the concrete in order to prevent the crushing of the pile. Concrete piles are quite often sunk by means of a water jet. This method is made possible by inserting an iron pipe in the center of the concrete.

Fig. 190 shows the details of the concrete piles employed in the construction of ore docks for the Pennsylvania Railroad



Courtesy of Raymond Concrete Pile Co.

FIG. 188.—Raymond pile-driving outfit, core swung in leads. Several sections of permanent shell on the ground.

Courtesy of Raymond Concrete Pile Co.

FIG. 189.—Raymond pile-driving outfit. Note the tapered core.

Company at Cleveland, Ohio. The piles, octagonal in shape and reinforced with eight 1-in round rods, were cast vertically in steel forms, a cast-iron shoe being fitted into the form and becoming a part of the finished pile. A 1:2:4 mixture of both gravel and broken stone was used. The forms were removed in

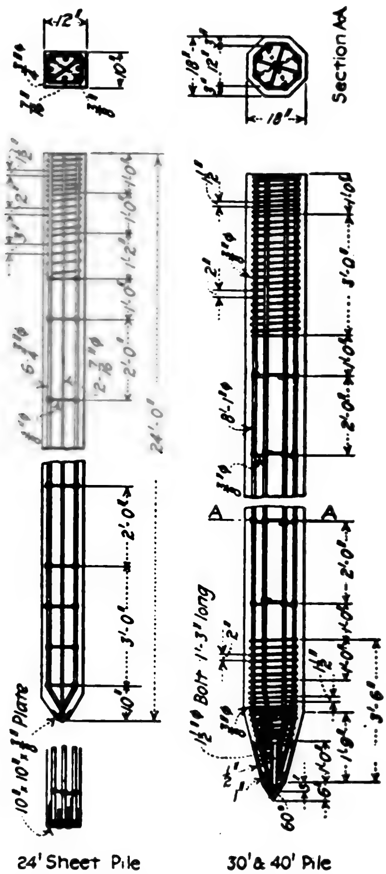


FIG. 190.—Details of piles used in ore dock construction at Cleveland.

from 12 to 24 hours after pouring, depending upon the weather. No pile was driven before it was at least 30 days old. Sheet piles were also used, as shown in Fig. 190.

In a paper by Sanford E. Thompson and Benjamin Fox which was read before the Boston Society of Civil Engineers and which

was also reproduced in the *Journal of the Association of Engineering Societies*, Vol. XLII, 1909, there is a complete analytical description of the design, construction, and driving of the concrete piles used in the pile foundations of the Boston Woven Hose & Rubber Co's power plant. The paper is in part as follows:

"The piles as designed by Mr. Fox were made 30 ft. 6 in. long, 14 in. square at the butt end, and in general 9 in. square at the tip. A number of soundings were taken at the site of the power house which indicated a fill of from 6 to 8 ft.; below this to a depth of 29 ft. 7 in. to 31½ ft. from the surface, fine sand and mud (practically all may be considered sand); and below the sand a clay hard pan was reached which was tested to a depth of 13 ft. These tests, together with a consideration of the requirements, determined the length of the pile.

Courtesy of Mr. Benjamin Fox.

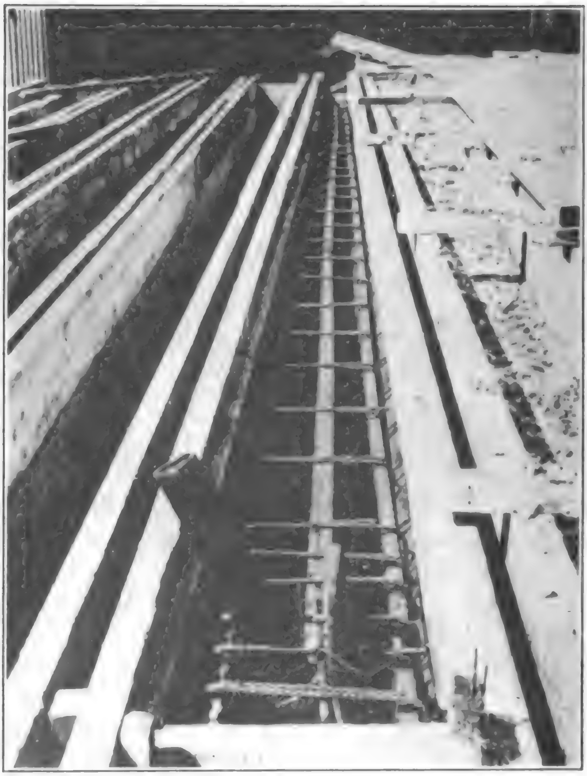
FIG. 191.—Pile reinforcement ready to place in form.

"Piles were reinforced with four ⅞-in. corrugated steel rods extending to within 2 in. of the ends of the pile, and embedded 2 in. from the face near the butt and 1½ in. from the face at the tip. Loops of ¼-in corrugated bars were placed around the principal steel, spaced about 12 in. apart, except near the butt where the spacing was decreased to 4 in., there being 34 loops in all. The butts of the piles were also extra reinforced, some with ⅜-in. and some with ½-in. rods, varying in length from 2 to 3 ft. A ⅞-in. rod about 5 ft. long was imbedded in the concrete with a loop sticking out through the concrete near the top of the pile on one side for hooking with derrick. Fig. 191 shows a view of the reinforcement wired up and ready to place in the form. Fig. 192 shows the reinforcement in the form ready for the concrete.

"A galvanized iron pipe was cast in the center of the pile for the water jet. For experimental purposes the sizes of pipes were varied, being 2 in., 1½ in., 1¼ in., and 1 in. To carry out the experiment still

further some of the piles were made with one of the larger-size pipes for about half the length of the pile and there connected with one of the smaller pipes which extended down to the tip.

"The friction of water running through pipe of small size is very great, so that it is known without experimenting that the largest-size pipe which is practicable to insert in a pile will give the least loss of head and



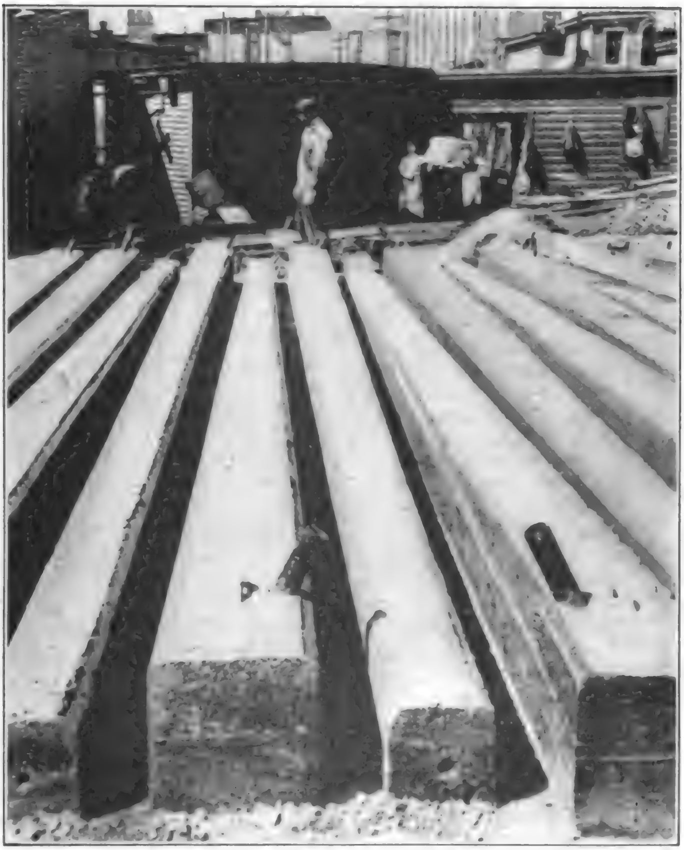
Courtesy of Mr. Benjamin Fox.

Fig. 192.—Form with reinforcement in place ready to pour concrete.

therefore be the best. To increase the velocity of the water, and thus increase its power to loosen earth (note that it is the velocity, not the pressure which is increased) the size of the tube should be reduced near the tip. The reduction must be made far enough from the pile to prle vent clogging under heavy blows. There is no danger of the nozze-filling while the water is flowing freely, and therefore no danger while the pile is being churned down in the first few blows. The danger is

apt to occur when the driving becomes hard, and the penetration per blow is small.

"A 2-in. pipe is probably as large as is practicable, and it is therefore suggested that this size be used to within 12 in., or if preferred, 24 in. from the tip, and there reduced to 1 in.



Courtesy of Mr. Benjamin Fox.

Fig. 193.—Finished piles on casting platform, showing handling hook, jetting pipe and the manner in which reversed ends economize space.

"The method of construction is as follows: A 2-in. platform of rough plank is built on ground of sufficient area to hold all of the piles. On this platform chalk lines are struck and V-strips to form a 1-in. chamfer nailed so that the lines of the piles are about 6 in. apart, alternating

points and butts. The casting of the piles with tips and butts alternating is economical of space, but where the piles are cast so as to be handled directly by the pile driver, without any intermediate handling, it is best to cast them all with the butts toward the machine on account of the saving of time in getting the pile in the gins. Two 8-in. unplaned (to assist in skin friction) spruce planks form the sides of each pile. The piles are made in lots of about five. The outside form is cleated with 2 by 4's and the other sides have the plank simply set on edge with pairs of wedges between them. There are 7 cleats or wedges

Courtesy of Mr. Benjamin Fox.

FIG. 194.—Completed piles after driving.

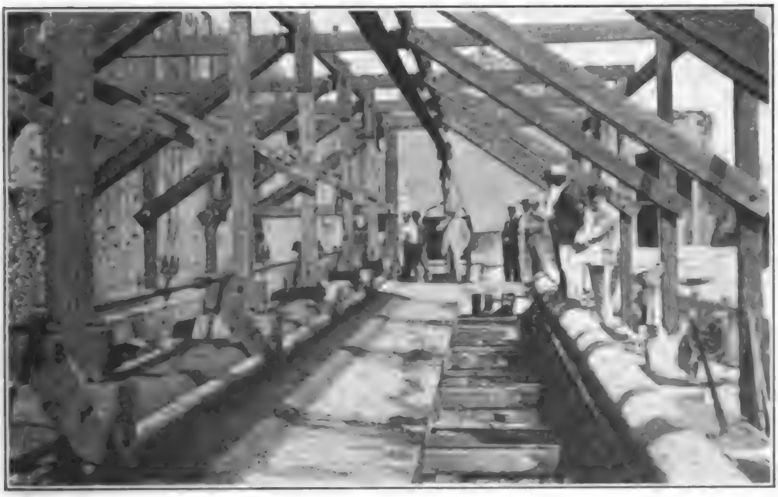
in the length, and 7 pieces 2 by 4 nailed across the top of each. After setting the plank sides, beveled pieces are nailed to locate the upper surface and form a chamfer. The reinforcement when made is suspended in form by 2 wires attached to each of the 2 by 4 cross-pieces.

"Fig. 193 shows a view of piles with the forms removed. Two days were usually allowed before striking the forms. Fig. 194 shows the completed piles after driving.

"The concrete was mixed by hand in the proportion of 1:2:4, using $\frac{3}{4}$-in. trap rock, the sand and cement being first made into a mortar and the stone added. A thorough mix is, of course, essential. Mixing

was started in March, precautions being taken at night against possible frost and the piles wet down every day for 2 weeks.

"The age of the piles when driven ranged from 30 to 41 days, the larger part of them being nearer the shorter age. The first pile was molded on March 24 and driven on April 24, and during this period the temperature was low, averaging between 40° and 50° F., so that the piles had not attained nearly their full strength. After the driving was commenced the weather became much warmer, and the piles after the first few were noticeably harder and entirely satisfactory, even although



Courtesy of Mr. A. C. Chenoweth.

FIG. 195.—"Chenoweth" machine, used in casting rolled concrete piles. Pile being rolled.

the age was practically the same, that is, about 1 month. It may be said that a period of 1 month for seasoning piles is sufficient during, say, the months between May 1 and October 1, but during the colder months a longer period should be alllowed unless artificial heat can be used to hasten hardening."

There have been a number of inventions in the concrete-piling field developed along the lines of the *made-up* pile. The principal systems are the Gilbreth and the Chenoweth.

Gilbreth Pile.—The Gilbreth pile is a corrugated reinforced-concrete pile tapered so as to develop the full bearing power of the soil throughout its entire length. A hole is made through the center of the pile to receive a 2-in. jet pipe, which is used to sink (or assist in sinking) the pile.

Chenoweth Pile.—The following is taken from an article in *The Journal of the American Society of Engineering Contractors,* April, 1912, written by Mr. Alexander C. Chenoweth, inventor of the *Chenoweth* concrete pile:

"The Chenoweth concrete pile is of what has been termed the molded type, but differs materially from other piles of that class by being rolled into shape instead of cast. The process is shown in Fig. 195.

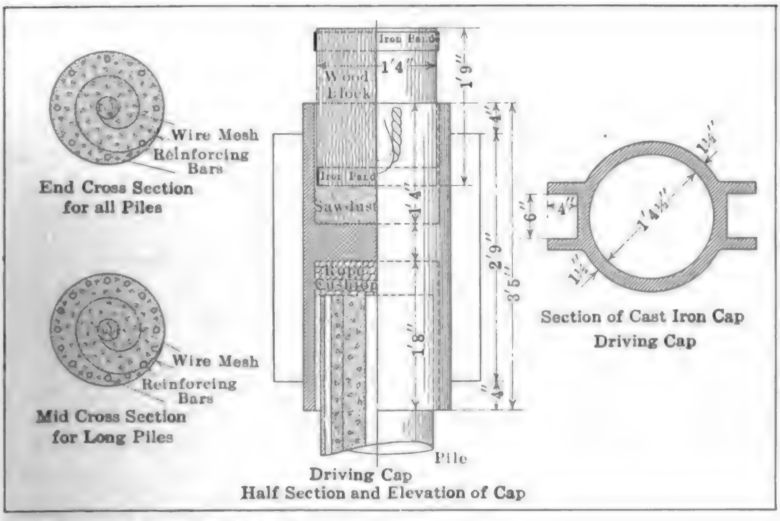
Courtesy of Mr. A. C. Chenoweth.

FIG. 196.—View of "Chenoweth" machine, showing wire mesh placed on movable table and attached to pile mandrel.

"This pile is made by the machine illustrated in Figs. 195 and 196, which consists of a platform over which is suspended longitudinally the mandrel around which the material is rolled, and above it the heavy longitudinal roller that aids in compressing the materials while being rolled.

"On the platform is placed a sheet of wire cloth of the proper dimensions. This is then attached to the mandrel and coated with a layer of gravel concrete 2 in. or more in thickness, in which are embedded the longitudinal reinforcing rods. The mandrel is now revolved and the concrete-covered sheet wound around it mechanically. The heavy roller, resting upon it all the time, insures a uniform and compact pile.

"At the completion of the rolling, the pile rests on a series of rollers at the edge of the platform. It is then wrapped around every eight or more inches with wire, to prevent if from unrolling, on the completion of which the pile is rolled on to the cars and taken to the seasoning yard.



Courtesy of Mr. A. C. Chenoweth.

Fig. 197.

At no time during this formative period is it necessary to lift the pile and thus produce any disturbance of the materials of which it has been made.

"The concrete when applied to the wire cloth is moderately wet and is sprinkled with water. When the compression rolls come into operation the surplus water oozes out. The resultant material is very dense.

"The machine can be set up when the piles are to be used or it can be located at some central point and the piles transported to the work. One strong feature of this pile is that it can be made in very long lengths.

The general scheme of the reinforcement admits readily of modifications and is, therefore, adaptable to the various conditions met with in concrete pile work. In some form of dock construction the pile is unsupported for a long distance and must be sufficiently reinforced to meet this condition. This pile is driven or jetted into place as the situation demands. The driving cap or cushion is shown on Fig. 197."

CHAPTER IX

WALLS AND PARTITIONS

53. Bearing Walls.—As suggested at the beginning of this Section, the two principal types of reinforced-concrete buildings differ only in the character of the outside walls. A skeleton form of construction is assumed in most of the discussions in this text, but bearing-walls should have some consideration.

Although the bearing-wall type of construction is naturally the only type employed in concrete residences, it has not been found in much favor for manufacturing and commercial buildings in general. Of course, a building with monolithic wall construction possesses great rigidity when properly designed and reinforced, but it cannot be erected as rapidly as buildings with other wall types, and, unless great care is employed in construction, there is a likelihood of defects occurring in casting the work. There is also difficulty in obtaining a uniform color for the wall surfaces.

Concrete walls are either of single thickness, or of double thickness with an air space between. A common method of obtaining an air space is by building in a central course of hollow-tile blocks. Double walls render the interior of the building less subject to changes in temperature and more completely moisture-proof, but, in the majority of commercial and manufacturing buildings, single walls, 6 in. or more in thickness, are not objectionable and make a great saving in floor space. Occasional projections or pilasters improve the appearance and add to the strength of a single wall. Fig. 198 shows the front elevation of a warehouse for the Portland (Me.) Savings Bank which, for the bearing-wall type of structure, is quite ornate. Note the cornice, belt courses, etc.

Except in residences, bearing-wall footings must usually be reinforced. A cantilever projection is formed on each side of the wall and the amount of reinforcement may be determined as for a simple cantilever beam.

Sometimes only portions of a wall are made to carry floor

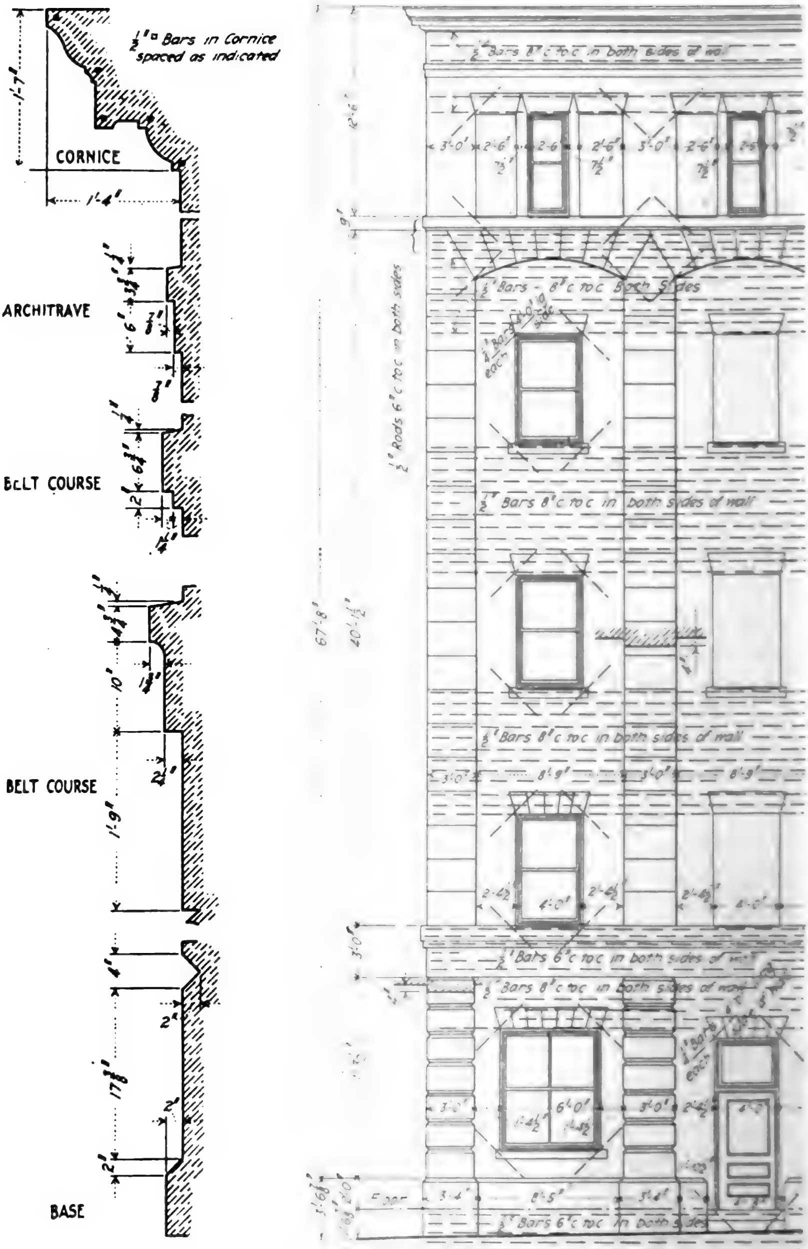


Fig. 198.—Front elevation of warehouse for Portland Savings Bank, Portland, Maine.

loads. Walls of this kind are shown in Figs. 181C, 181D, 199A, and 199 B.

In designing the bearing wall shown in Fig. 181D, Section BB, the offset was made 12 in. on the inside in order to carry a 12-in. brick wall above. The 8 in. on the outside is to allow for bearing for floor slab in future extension. The wall had originally been laid out for a thickness of 16 in., and the footing was placed with this in mind. When it was decided to make the wall thicker, it was impossible to change the location of the footing so as to center it, but the difference was so small it was not considered important. The walls were reinforced horizontally between piers to take care of the earth pressure—the idea being to make the pier act as a beam, supported by the basement and first floor.

The footings rested on a very soft rock. It was the belief of the designers that the wall was sufficiently deep to act as a beam and to distribute the load from the piers over the entire bay from column to column. This wall was cast integrally with the piers and was reinforced with four $1\frac{1}{8}$-in. rods near the top as shown. The footings then were reinforced in one direction with the same spacing of rods from column to column.

54. Curtain Walls.—In the usual type of construction, wall girders are placed at each floor and the reinforced-concrete walls (called *curtain* walls) are designed merely to fill in the panels between the columns and girders which form the skeleton frame of the building. They are not intended to carry any weight but need to be strong enough to withstand pressure, which is usually considered to be 30 lb. per square foot. Such walls may be designed as slabs carrying a uniformly distributed load and supported on all four sides. Figuring in this way, using the above value for the wind pressure, a wall in ordinary building construction will never exceed $3\frac{1}{2}$ to 4 in. in thickness, which is the practical limit for ease in construction. In fact, 6-in. walls will generally be required in order to obtain sufficient imperviousness to moisture. Also, a 6-in. wall is usually cheaper than a 4-in. wall owing to the greater ease of pouring the concrete and placing the reinforcing steel in position. A 4-in. wall, however, is all that is needed for fire protection.

In addition to the consideration of wind pressure, concrete walls are reinforced generally for the purpose of preventing cracks and guarding against accidents during or immediately after

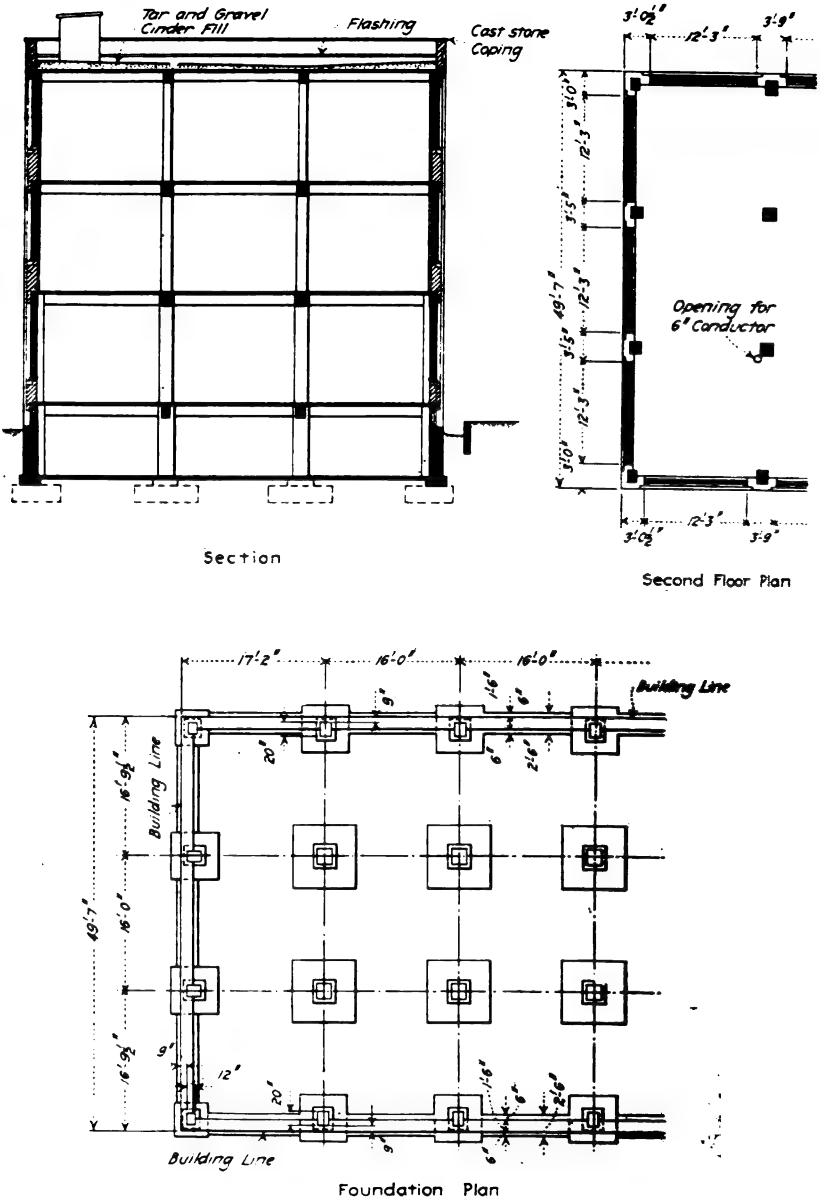


FIG. 199 A.—Factory addition, Bradley Knitting Works, Delevan, Wis.

construction, except, of course, where lateral pressures occur, when the beam action must be considered. The reinforcement generally consists of ¼- to 1¼-in. rods placed both horizontally and vertically from 12 to 24 in. apart.

Slots in the columns are made by nailing a strip on the inside of the column forms. In this way the curtain walls are mortised

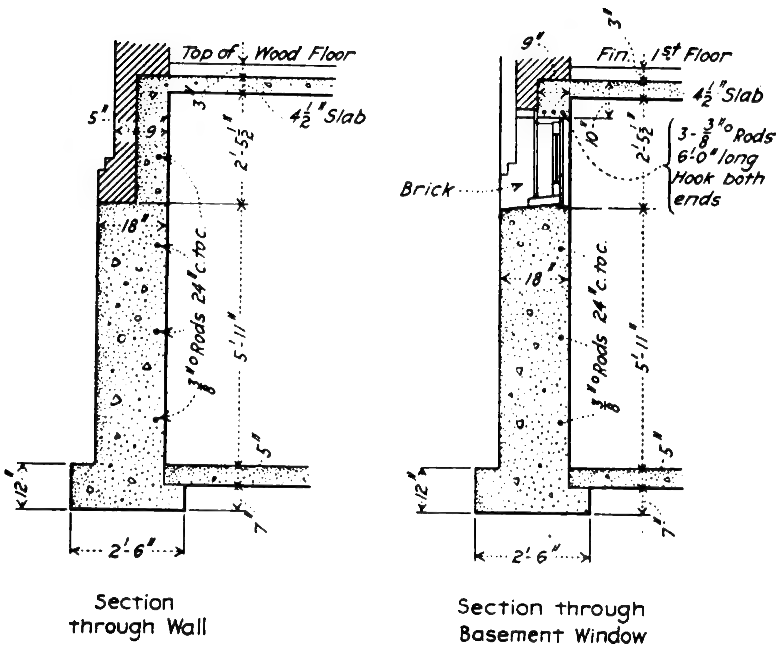


FIG. 199 *B*.—Factory addition, Bradley Knitting Works, Delevan, Wis.

into the columns and a contraction joint is formed, so that the contraction to be provided for is due only to the curtain wall itself.

When curtain walls are employed with a small percentage of window area, it is customary to use both horizontal and vertical reinforcement, the same as when the wall covers the entire panel, and to place one or more rods near the edge of the slab about all openings. When wire fabric is used for reinforcement, the edges of the openings are stiffened by bending back the fabric into a U shape.

Where a large percentage of the outside wall of a building is used for windows, as is generally the case in factory buildings, the wall proper consists of little more than columns with deep wall beams or wall girders forming belt courses between them. The spandrels, which we shall call the walls directly beneath the

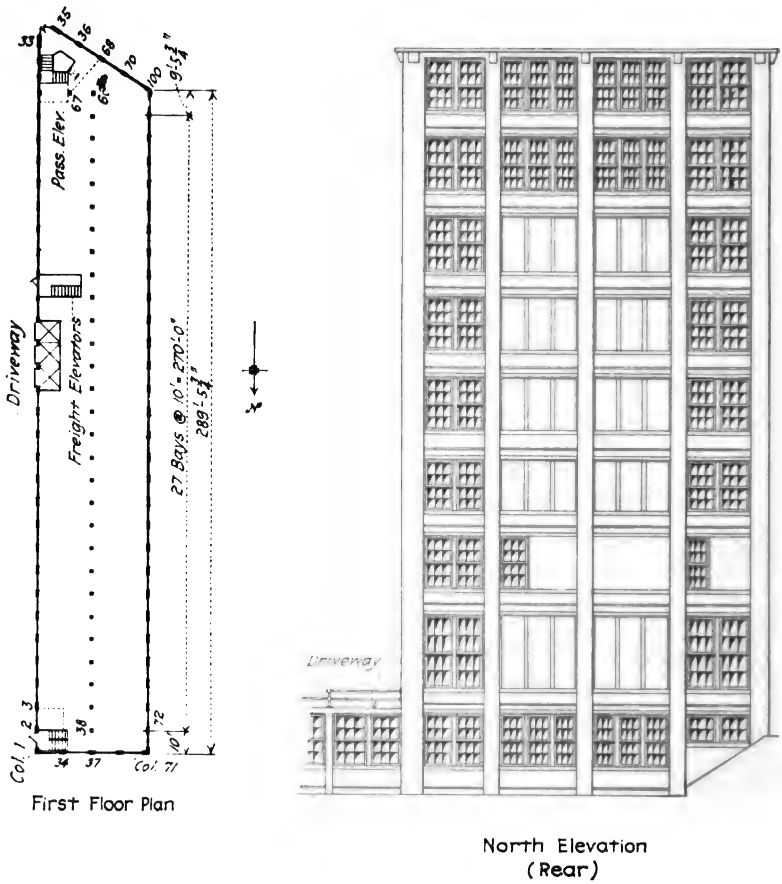


FIG. 200.—Lang building, Haverhill, Mass.

window sills, may be reinforced so as to act as the upper part of the wall beams, but the usual method is to consider this portion as separate from the beam and merely reinforce with small rods (in some cases with wire fabric) so as to prevent cracks and to reinforce the construction during the setting of the concrete. If this is done, the spandrels may be put in after the main struc-

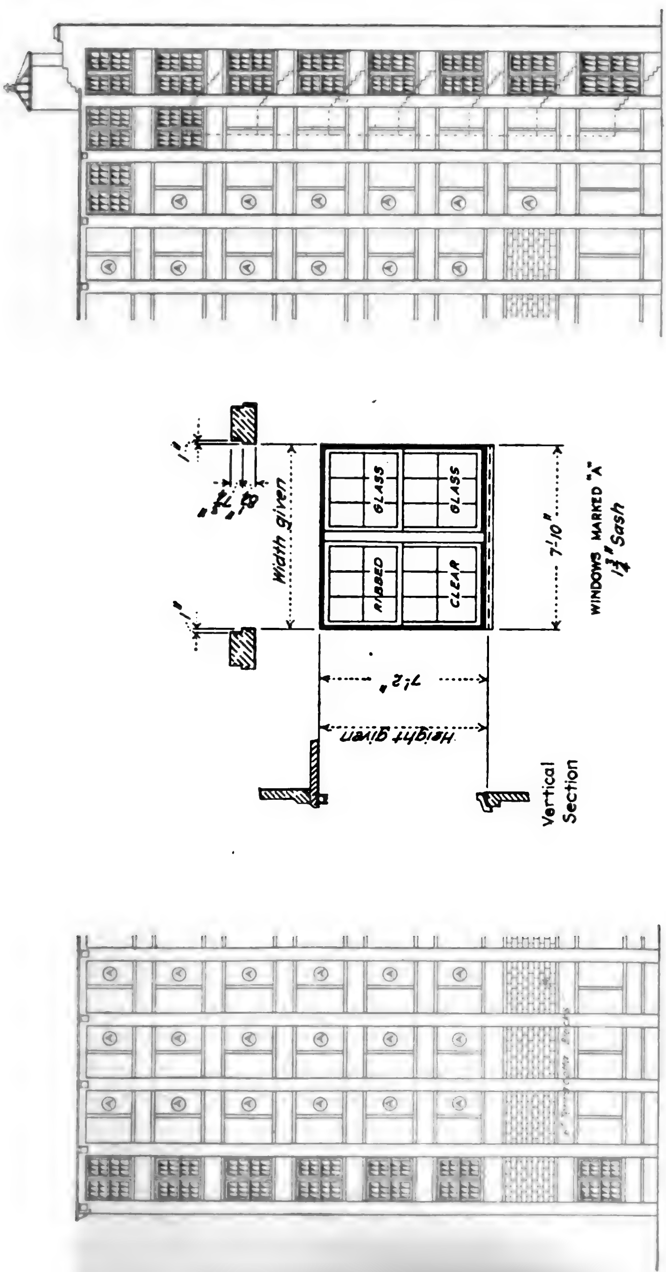
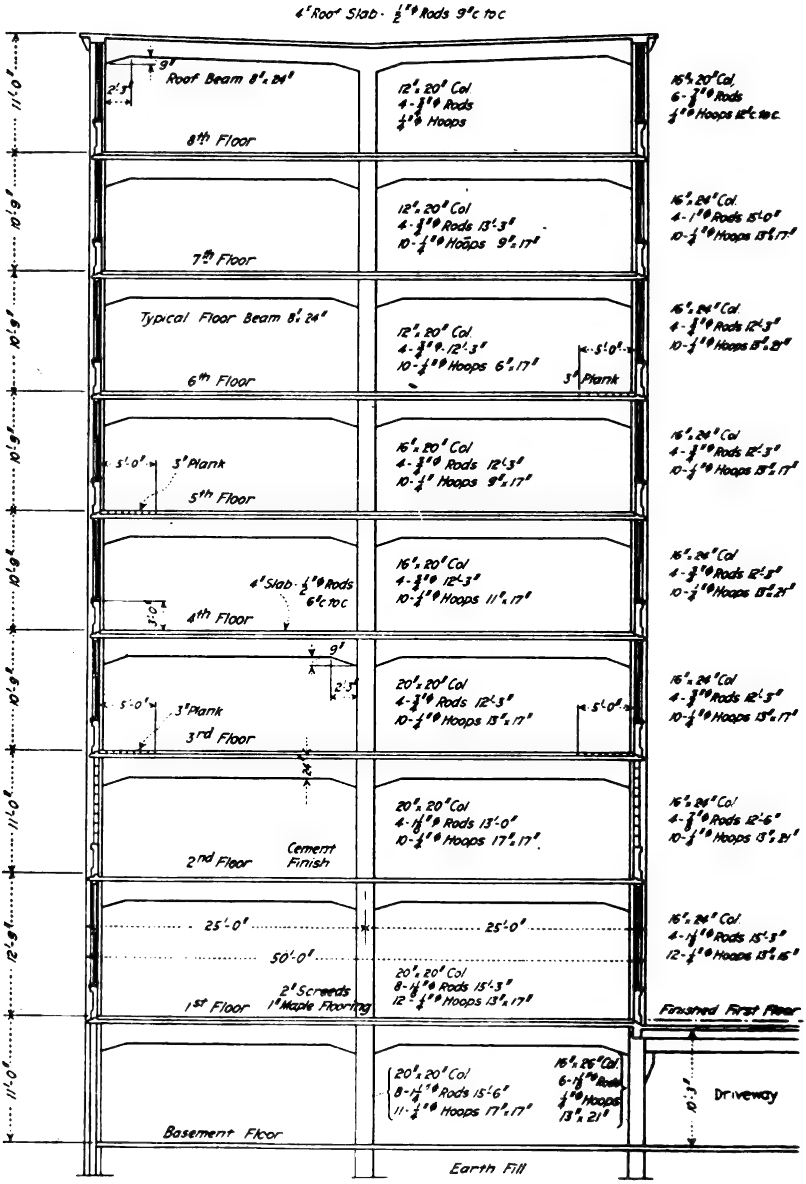


Fig. 201.—Lang building, Haverhill, Mass.



TYPICAL CROSS SECTION THROUGH BUILDING

FIG. 202.—Lang building, Haverhill, Mass.

tural parts have been cast, in the same manner as complete curtain walls. This saves time in the erection of the building and allows the use of more care in obtaining a neat finish on the spandrel walls.

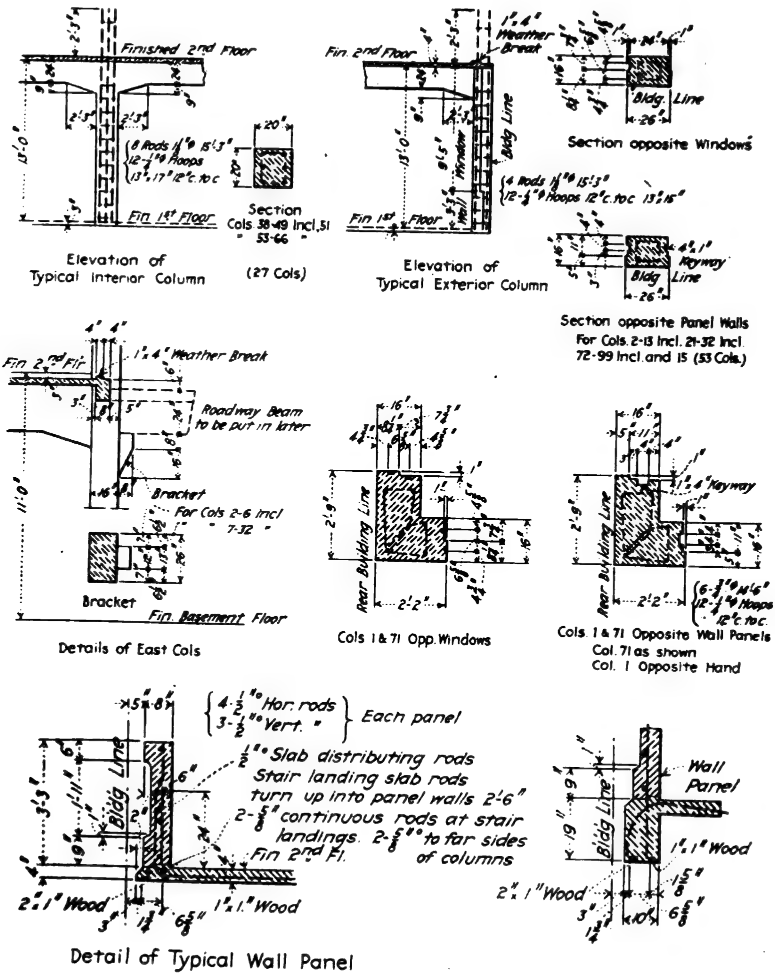


FIG. 203.—Details of Lang building, Haverhill, Mass.

In Fig. 203 two spandrel sections are shown, one where wall beams are employed and the other where only the floor slab and spandrels form the exterior belt course. It may be stated here that Figs. 200 to 207 inclusive show many of the details of

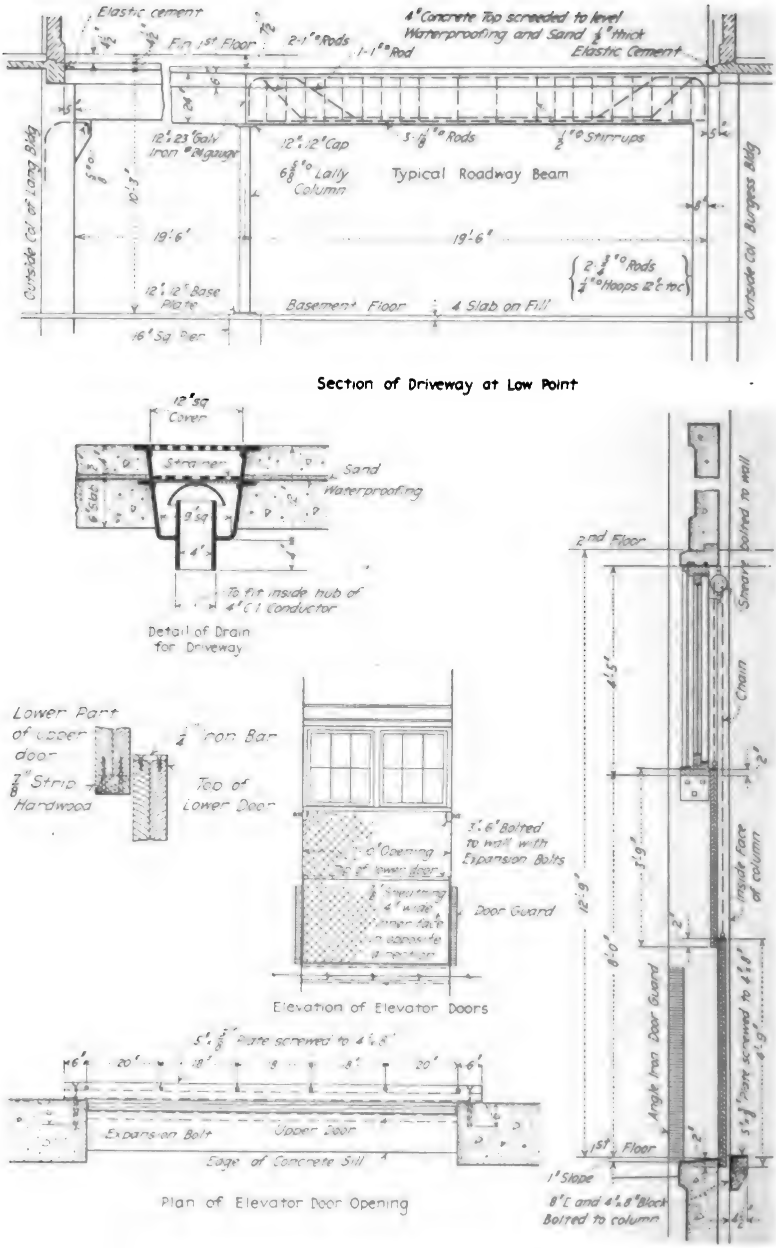


FIG. 204.—Details of Lang building, Haverhill, Mass.

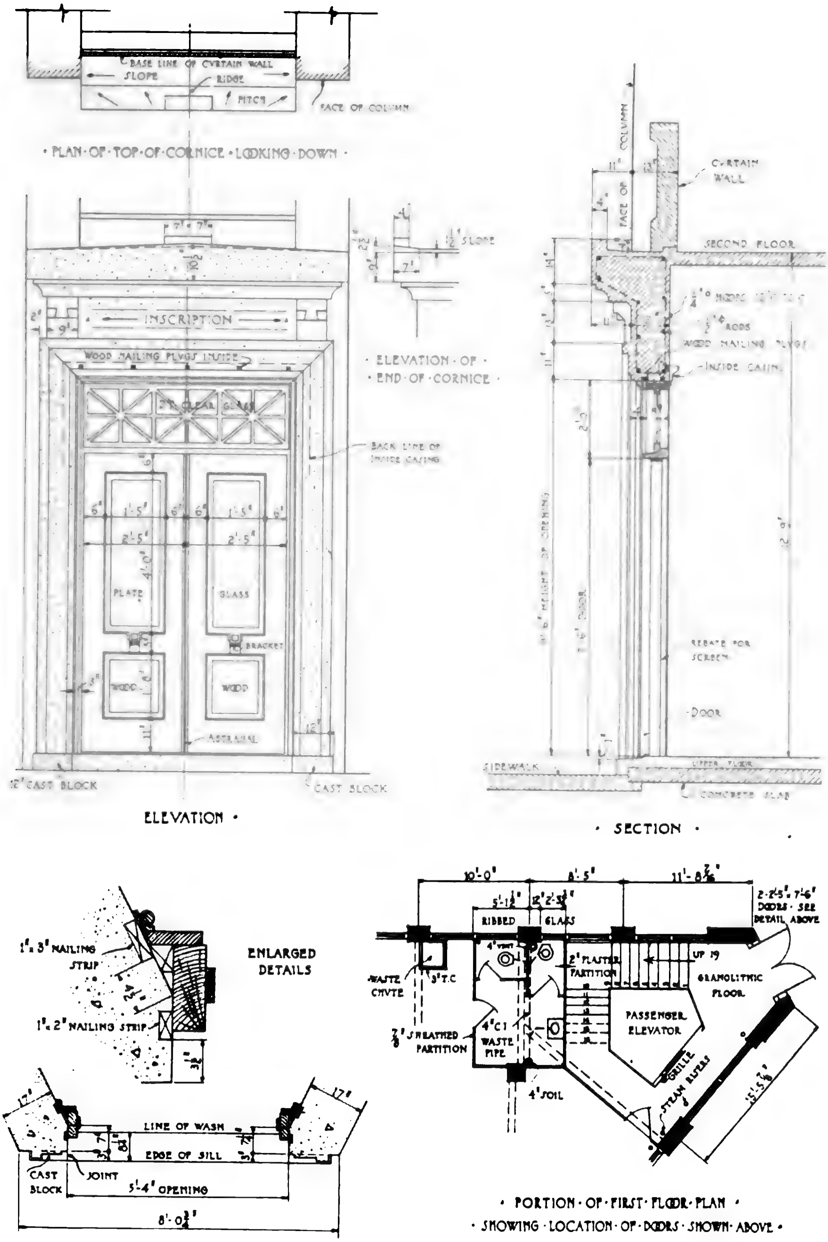


Fig. 205.—Details of Lang building, Haverhill, Mass.

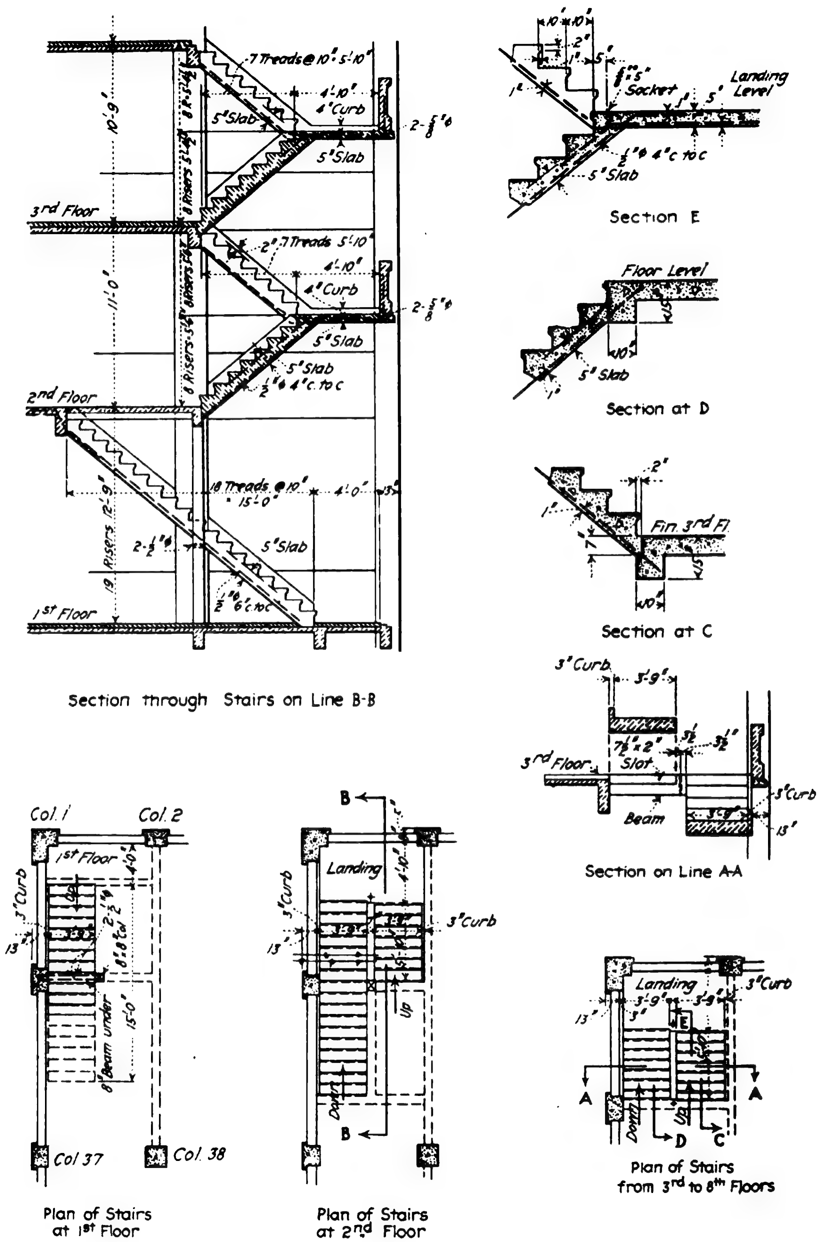


FIG. 206.—Details of Lang building, Haverhill, Mass.

the Lang Building at Haverhill, Mass. built by the Concrete Engineering Co. for the Haverhill Building Trust. Many of the details shown will be referred to later, but the student should

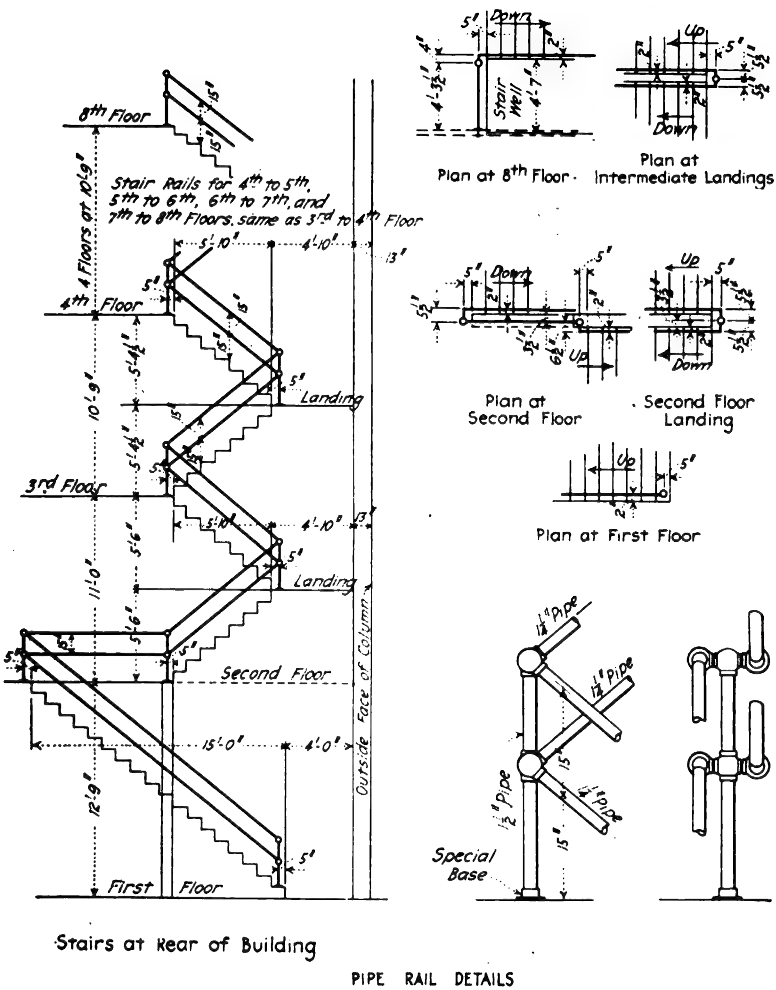


FIG. 207.—Details of Lang building, Haverhill, Mass.

look through all the details at this time so as to become somewhat familiar with them. Note, for example, the elevations of the building and its general shape; also the nailing strips in the columns and lintels.

Courtesy of Mr. C. A. P. Turner.

FIG. 208.—International Harvester building, Fort William, Canada.

Courtesy of Mr. H. F. Porter, Mfg. Editor of "Factory."

FIG. 209.—Warehouse, Dodge Mfg. Co., Mishawaka, Ind.

Reinforced concrete is well adapted to the construction of walls where considerable strength is required, but in very thin walls, such as curtain walls, it becomes a relatively expensive building material on account of the cost of forms. Brick, concrete blocks, or terra-cotta are often used on this account, not only for spandrel walls, but to cover entire wall panels (Figs. 208 and 209).

For curtain walls in *Unit* construction, reference should be made to Figs. 78 and Plate XI.

55. Brick and Other Veneer.—From an architectural standpoint it sometimes becomes necessary to cover the exterior

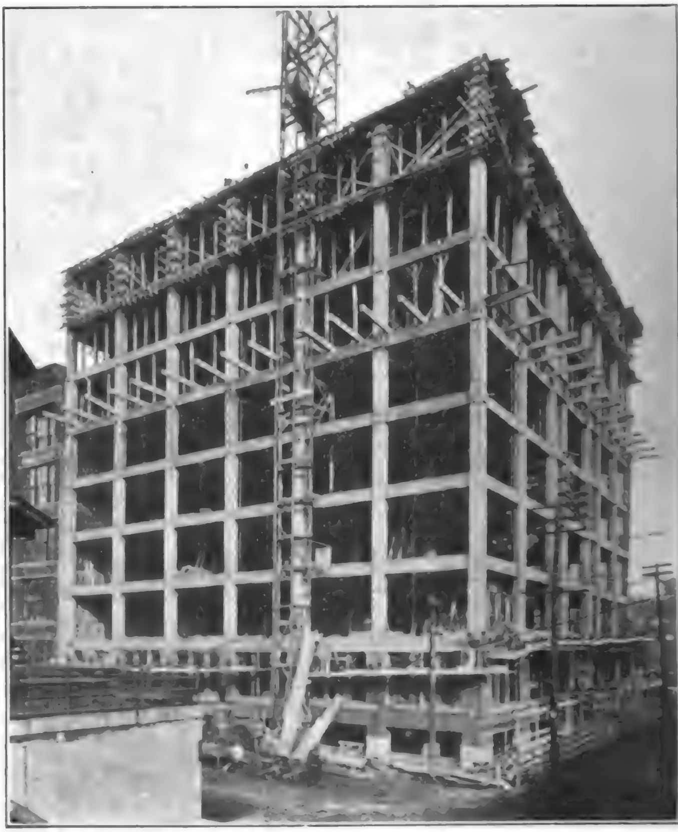
Courtesy of Universal Portland Cement Co.

Fig. 210.—J. K. McIntyre building, Dayton, Ohio.

columns and walls with brick, terra-cotta, marble, limestone, or other material. The facing or veneer is usually laid after the structural frame is completed (Figs. 210 to 212 inclusive).

A brick veneer generally consists of one thickness of brick, and this is tied into the concrete work by means of copper or galvanized-iron ties. These ties are usually about $\frac{3}{4}$ in. wide and are tacked to the form boards in such a manner that about 4 in.

of the tie will be embedded in the concrete work. When the forms are removed, the portion of the tie lying flat against the forms is bent out as shown in Fig. 213. A tie should be provided for about every 4 sq. ft. of wall surface. Brick facings should be supported, at least at every floor, by a ledge formed in the con-



Courtesy of Universal Portland Cement Co.

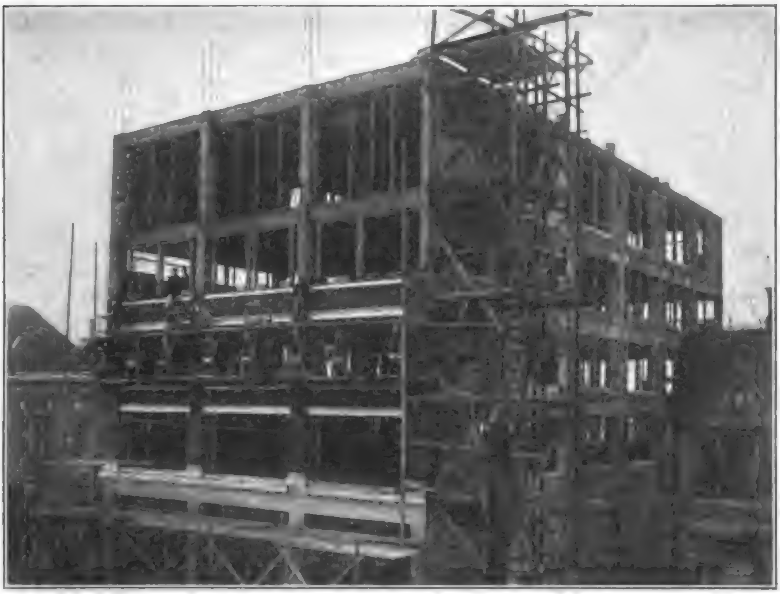
Fig. 211.—Rogers and Hall Co.'s building, Chicago, Ill.

crete. Angle irons are usually employed to carry stone and brickwork over door and window openings (Fig. 214).

Fig. 215 shows one method of supporting a stone lintel. The anchor at the top should be noted.

The arrangement of the ties and supports for terra-cotta varies greatly with the design. The facing is usually supported by

projections in the concrete fitting into openings in the terra-cotta, forming a sort of dovetailing. Iron anchors then tie the



Courtesy of Universal Portland Cement Co.

Fig. 212.—Primus building, Hammond, Ind.

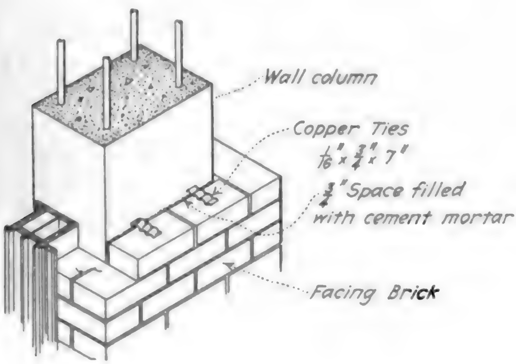
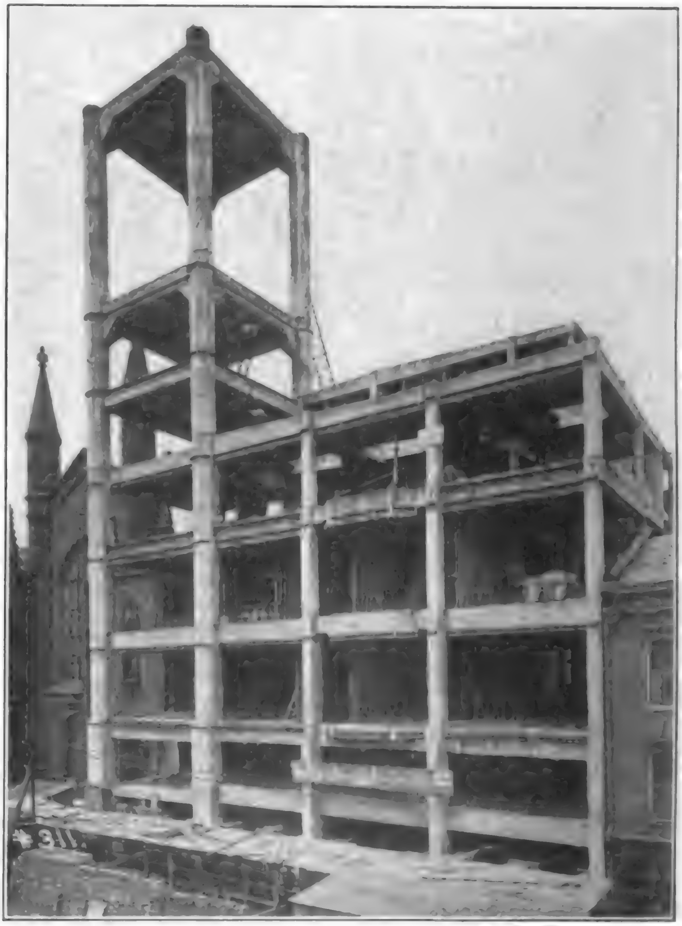


Fig. 213.

two together. It is very important that suitable play be provided for, as terra-cotta cannot be made to exact dimensions. Stone facings are also supported by projections in the concrete,

the same as terra-cotta, but the stone can be made to more nearly fit any simple shape given to the concrete ledge.

56. Window Openings.—The arrangement of floor slab and wall beam in Fig. 215 makes it possible to obtain the maximum



Courtesy of Ferro-concrete Construction Co.

Fig. 214.—Christ Church parish house, Cincinnati, Ohio.

amount of light within the building; that is, the window frames are run as close as possible to the underside of the floor slab. It should be noted that to accomplish this, the beams should either be run parallel with the walls of the building, so that none

of the beams will take bearing on the lintel, or else the floor arrangement should be similar to that shown in Fig. 202 where only parallel girders are employed spaced relatively close together. If beams run parallel with the walls, the spacing of the beams may be returned at the corners by a diagonal girder as

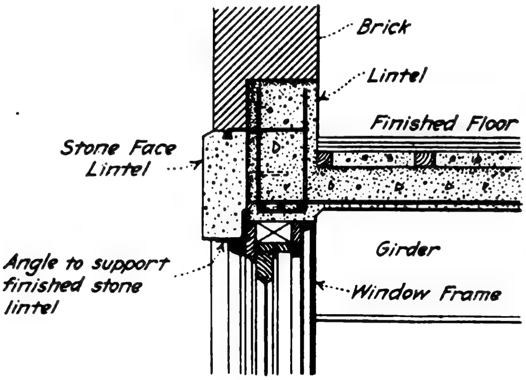


FIG. 215.

illustrated in Fig. 216. By so doing the windows may be made the same height throughout.

Reinforced-concrete buildings to be considered fire-resisting should have windows of wire glass, held in metallic frames. Wire glass is either ribbed, rough, maze, or polished plate having

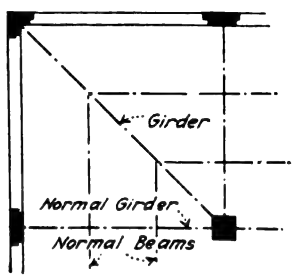


FIG. 216.

wire embedded in its center during the process of manufacture. The wire netting used for this purpose is similar to ordinary *chicken netting* with about a 1-in. mesh. It is embedded in the glass at a very high temperature which insures adhesion between the netting and glass, and the two materials become one and

inseparable. If the glass is broken by shock or by intense heat, it remains intact. It is this property which gives wire glass its fire-retarding qualities.

Metal frames are made in a great variety of forms to meet all purposes. Sashes may be stationary, sliding, pivoted either horizontally or vertically, hinged, or double hung with weights like an ordinary window. Sash may also be obtained which

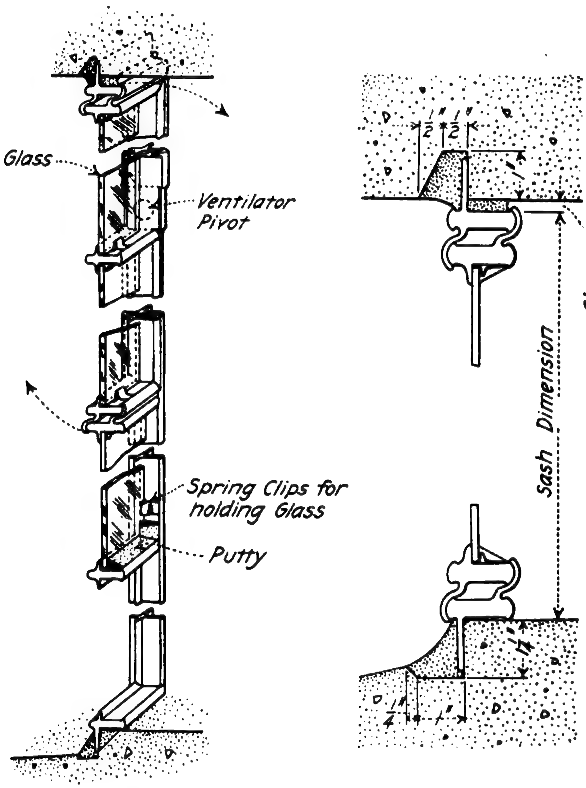


Fig. 217.

will close automatically and lock under fire by the fusing of a link or other means to accomplish the same result. A number of sections of steel-sash windows are shown in Figs. 217, 218, and 219. These sections are included in this text to show a few of the different methods of fastening metal frames to the walls or wall columns. Fig. 220 shows steel sash in place, ready for the placing of the sill. Fig. 221 shows the building completed.

When properly constructed, a tin-covered wood shutter is a most effective window protection. There is one objection, however, to the use of shutters on window openings and that is, they must necessarily be open while the building is in use, and, when the need comes for them, they are apt to be overlooked and not

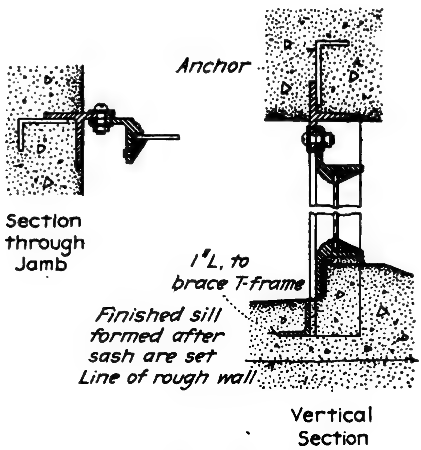


FIG. 218.

closed. They, moreover, hide a fire and are unsightly for many locations. Frames and sash of wood covered with metal are sometimes used—the wood furnishing the strength and the metal the protection. Note the combination of wood, tin, and steel in the window design shown in Fig. 222.

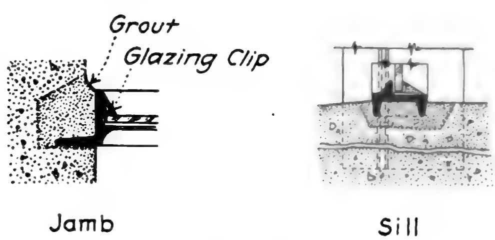
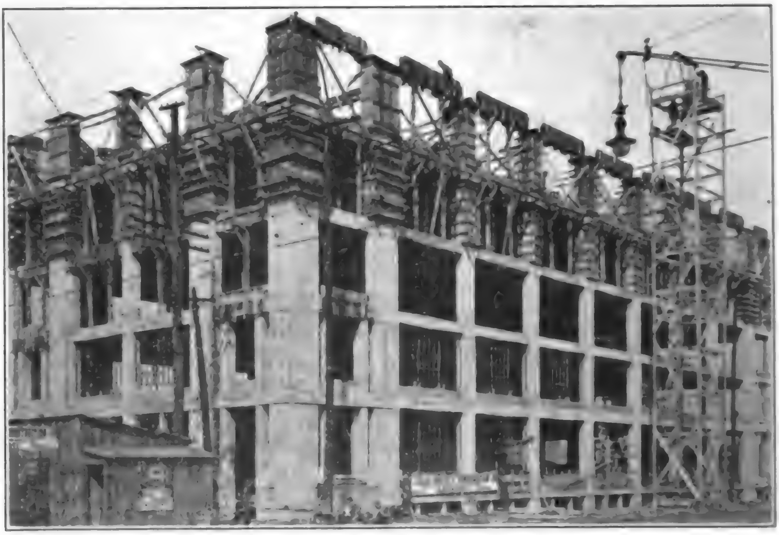


FIG. 219.

Automatic steel rolling fire shutters, placed either on the outside of window openings or in the window reveals immediately in front of the window frames and sash, form the least objectionable appearing type of shutter protection for windows, and seem



Courtesy of Ferro-concrete Construction Co.

FIG. 220.—Watkins medical building, Memphis, Tenn.

Courtesy of Ferro-concrete Construction Co.

FIG. 221.—Watkins medical building, Memphis, Tenn.

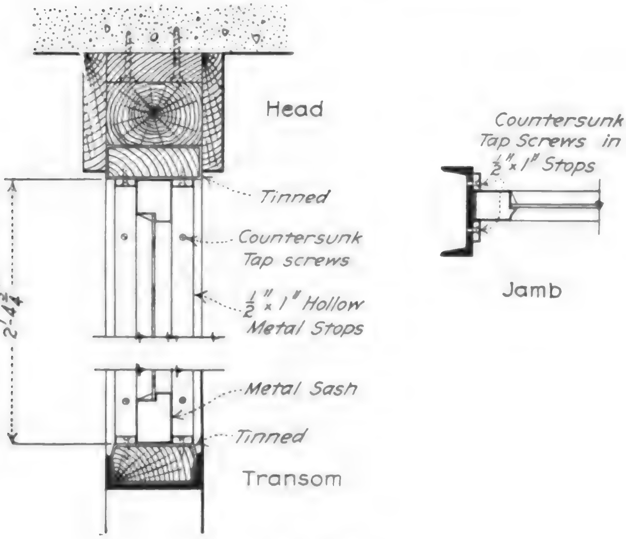


FIG. 222

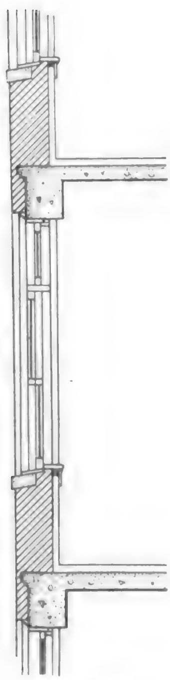


FIG. 223.

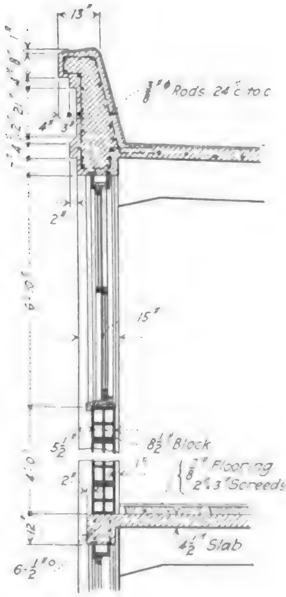


FIG. 224.

to be entirely adequate for all ordinary conditions. Open sprinklers, or *water curtains* as they are frequently called, are sometimes used in connection with shutters or wire glass windows, but do not seem to be very efficient in themselves except in very moderate exposures.

Fig. 223 shows a window construction with ordinary wooden sash employed in a large hospital in the Middle West. Figs. 201 and 202 show the details of window construction in the Lang Building at Haverhill, Mass. The nailing strips that must be placed in the concrete should be noted. Fig. 224 shows

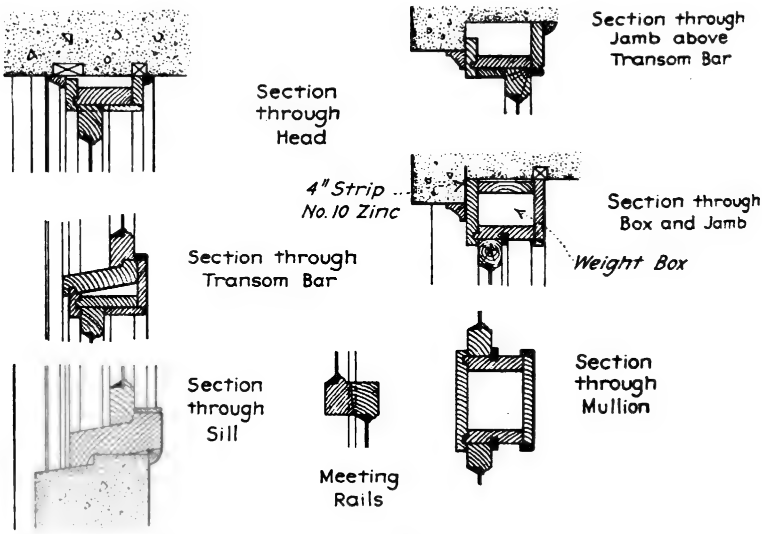


FIG. 225.

the window construction in a shoe factory at Cambridge, Mass. built for the Cambridge Building Trust. Double-hung window details for a building at the corner of Commonwealth Avenue and Beacon St., Boston, Mass., which we shall call the Cross-Roads Building, are shown in Fig. 225. A detail of a basement window opening is given in Fig. 199B. Fig. 226 shows a common window and wall construction. The details are those of the Truck Garage, Pacific Mills, Lawrence, Mass.

57. Door Openings.—Doors in a reinforced-concrete building should be fire resisting, the same as windows, and may be of either the hinged, sliding, or rolling type. If tin-covered wood

doors are provided on openings in interior partition walls, they are generally hung on inclined tracks so that they will close

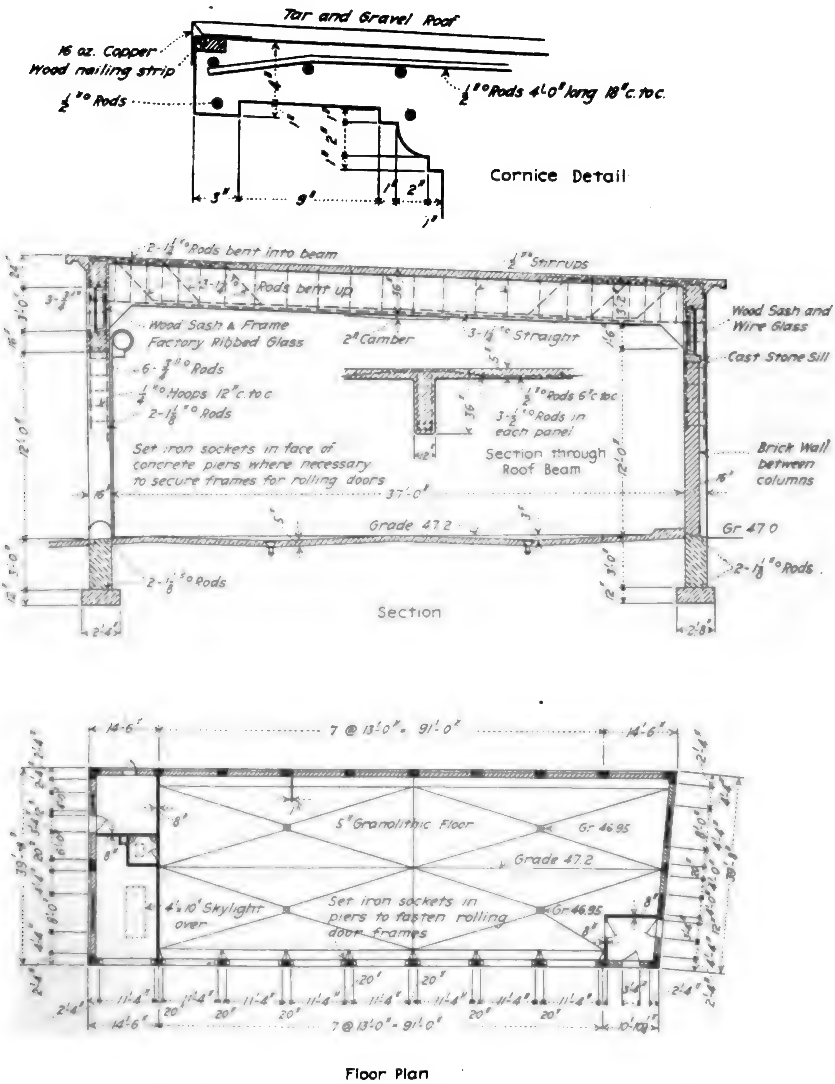


FIG. 226.—Truck garage, Pacific Mills, Lawrence, Mass.

automatically (Figs. 109 and 227). Where it is desirable to keep them open, an automatic release operated by a fusible link is provided.

Courtesy of Turner Construction Co.

Fig. 227.—Arbuckle Brothers' warehouse, Brooklyn, N. Y. Note vertically operated door at elevator shaft and sliding doors in the background.

Courtesy of Aberthaw Construction Co.

Fig. 228.—Interior cartridge storage building, Winchester Repeating Arms Co., New Haven, Conn. Live load, 800 lb. per square foot.

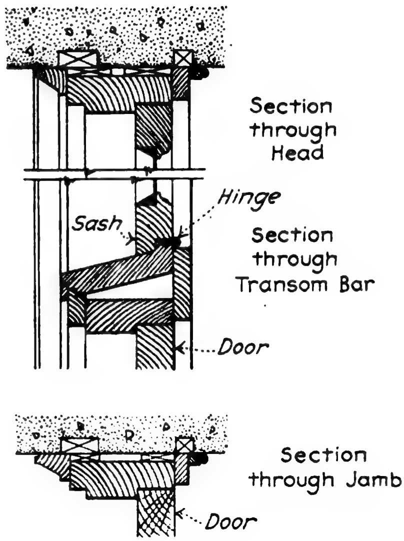


FIG. 229.

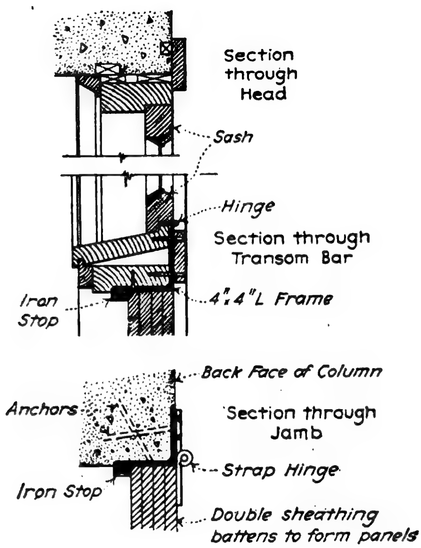


FIG. 230.

Sliding and rolling doors are generally used only for large openings such as driveway and freight-elevator entrances and are usually arranged to close automatically in case of fire. Sliding doors should have the rails on which they operate protected by metal shields to prevent obstruction. Steel rolling doors are made in horizontal, jointed, sectional strips, which wind up on a roller placed in a pocket above the opening, the ends moving in metal grooves to hold them in place (see *Frontispiece* and Figs. 226 and 228). There are a number of types of vertically operated doors. A common type for freight elevators is shown in Fig. 204.

Fig. 229 shows the details of an ordinary outside wooden door. Fig. 230 gives the details of large swinging driveway

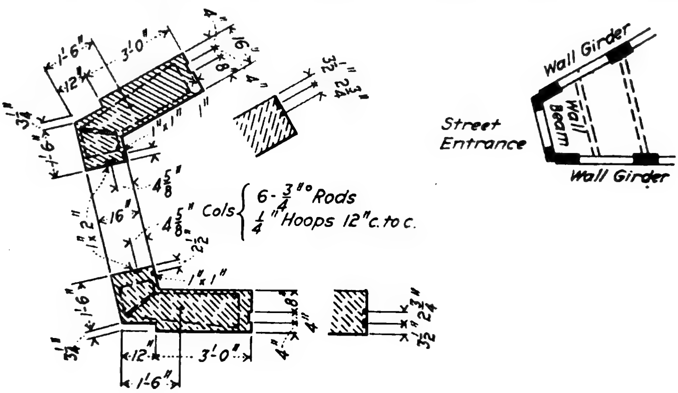


FIG. 231.

doors used in the Cross-roads Building at Boston, Mass. The arrangement around the main-street entrance doors to the same building is shown in Fig. 231. Similar door details in the Lang Building (Fig. 205) should also be noted.

58. Parapet Walls and Cornices.—Figs. 143 to 147 inclusive, 198, 199A, 224, and 226 give some idea of the possibilities in parapet wall and cornice construction.

59. Basement Walls.—Basement walls to sustain earth may be designed as simply supported slabs, the earth-pressure reactions being usually taken by the basement and first floors. The common construction is to employ curtain walls at least 8 in. thick between wall columns, and, in addition to reinforcing them vertically to take the earth pressure, to place rods near the bottom of the wall so as to make the wall carry itself as a beam from

footing to footing. This type of wall thus requires no foundation of its own and may be built in the same way as the curtain walls in the upper stories. Sometimes it becomes necessary to reinforce horizontally for the earth pressure. This brings a lateral force on the columns, but the resulting eccentricity of column loading may be disregarded for ordinary cases unless a very light wall column is used. It is customary in the design of basement walls to assume the earth pressure as due to a fluid weighing 30 lb. per cubic foot.

In Fig. 232 is given a complete basement plan of the Crossroads Building, Boston, Mass. The details of the basement walls are shown in Fig. 234. The foundations (Fig. 233) consisted of wells 3 ft. in diameter flared at the bottom and sunk to a good gravel subsoil. The top soil was of a peat formation and it was decided that the wells were about as cheap to build as spread foundations with piles. Steel tubes were employed 3 ft. in diameter which were withdrawn as the wells were filled with concrete. The wells were excavated by hand; a windlass on a tripod was placed over the hole and used to hoist the excavated material. A space 3 ft. in diameter is about the minimum area in which one man can work to advantage and that is the reason for the adoption of the 3-ft. wells. In nearly all the wells, the flare at the bottom was made without bracing of any kind because of the nature of the material. On account of the high banks, it was necessary to do considerable trenching along the street sides before the tubes could be started. Elevations of the ground surface before excavation was begun is shown in the location plot.

The building was erected up to the adjacent property line, and cantilever footings were employed. The basement walls on this side were figured for earth pressure since it appeared quite probable that the adjacent ground might be filled in at some future time. The floor designs of this building are quite complicated since the building is separated near its center by a 6-in, terra-cotta wall and the floors on each side are at different levels. A driveway also runs through the building at the first floor level just to one side of the separating wall, and, since the streets at this point are at different elevations, the driveway is on a grade. This explains to a certain extent the peculiar cross-sections shown for the basement walls. The basement columns and the entire first floor were finished before the basement was thoroughly

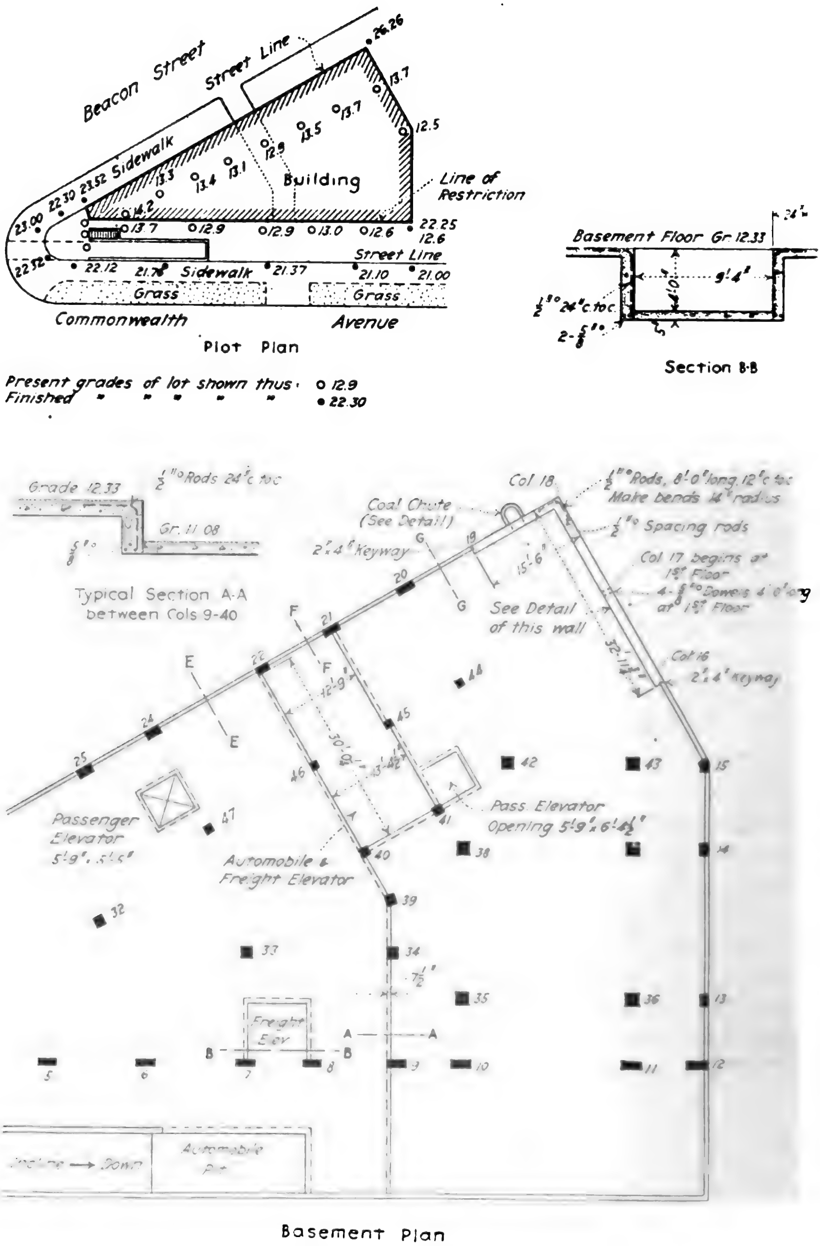


Fig. 232.—Cross Roads building, Boston, Mass.

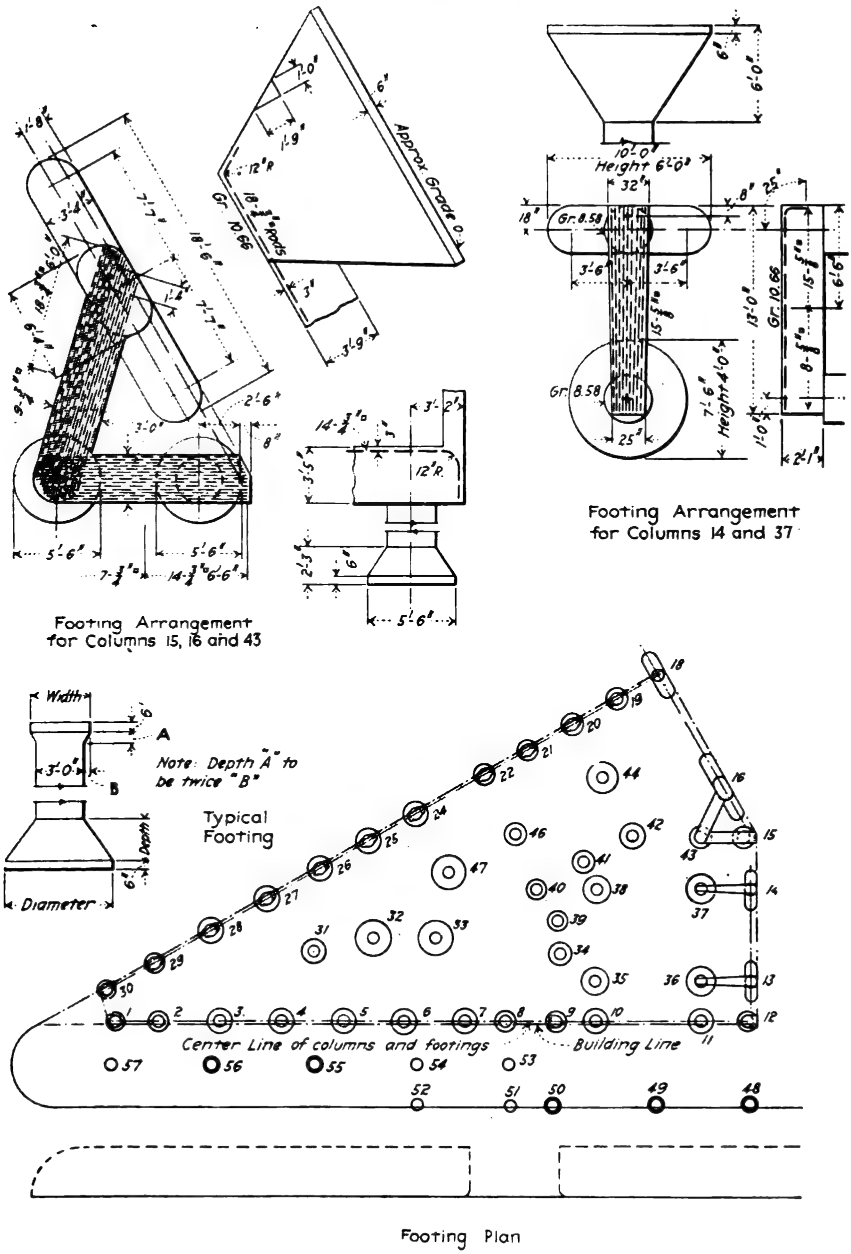


Fig. 233.—Cross Roads building, Boston, Mass.

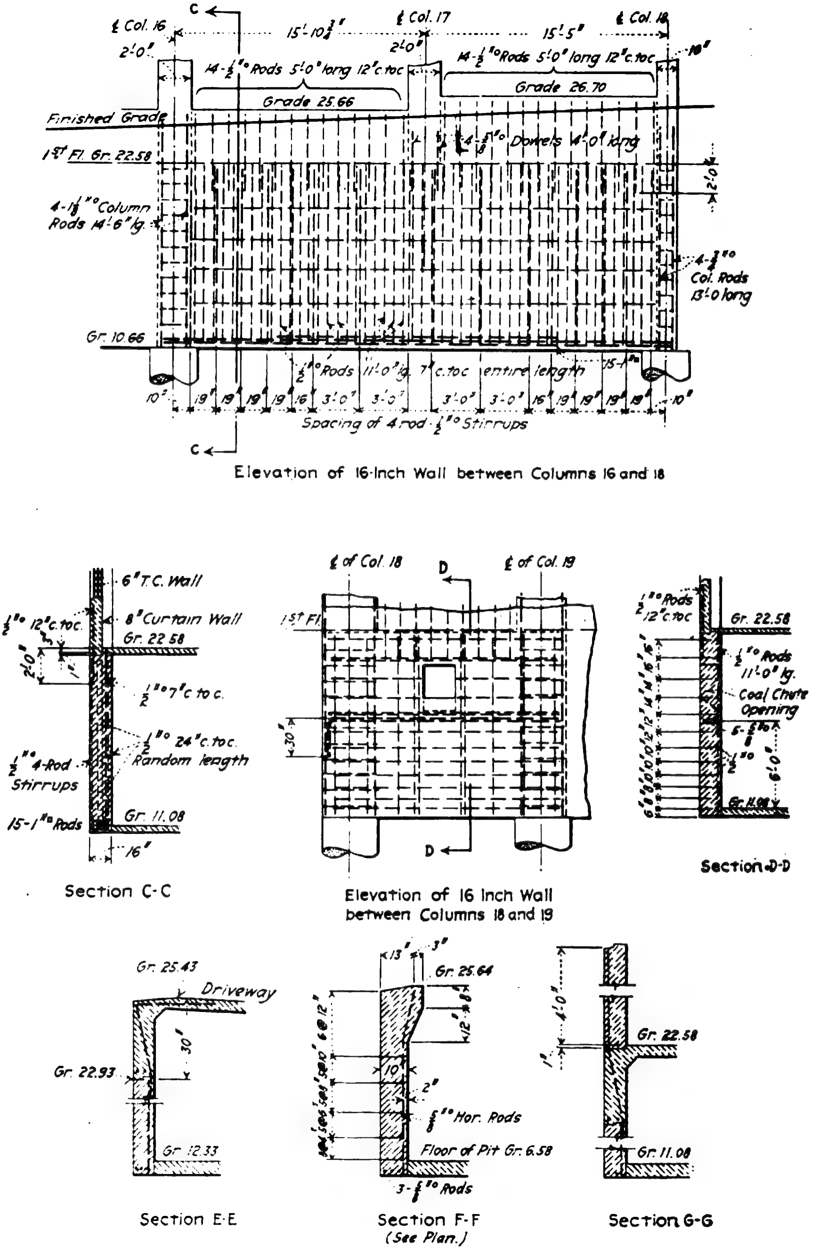


FIG. 234.—Cross Roads building, Boston, Mass.

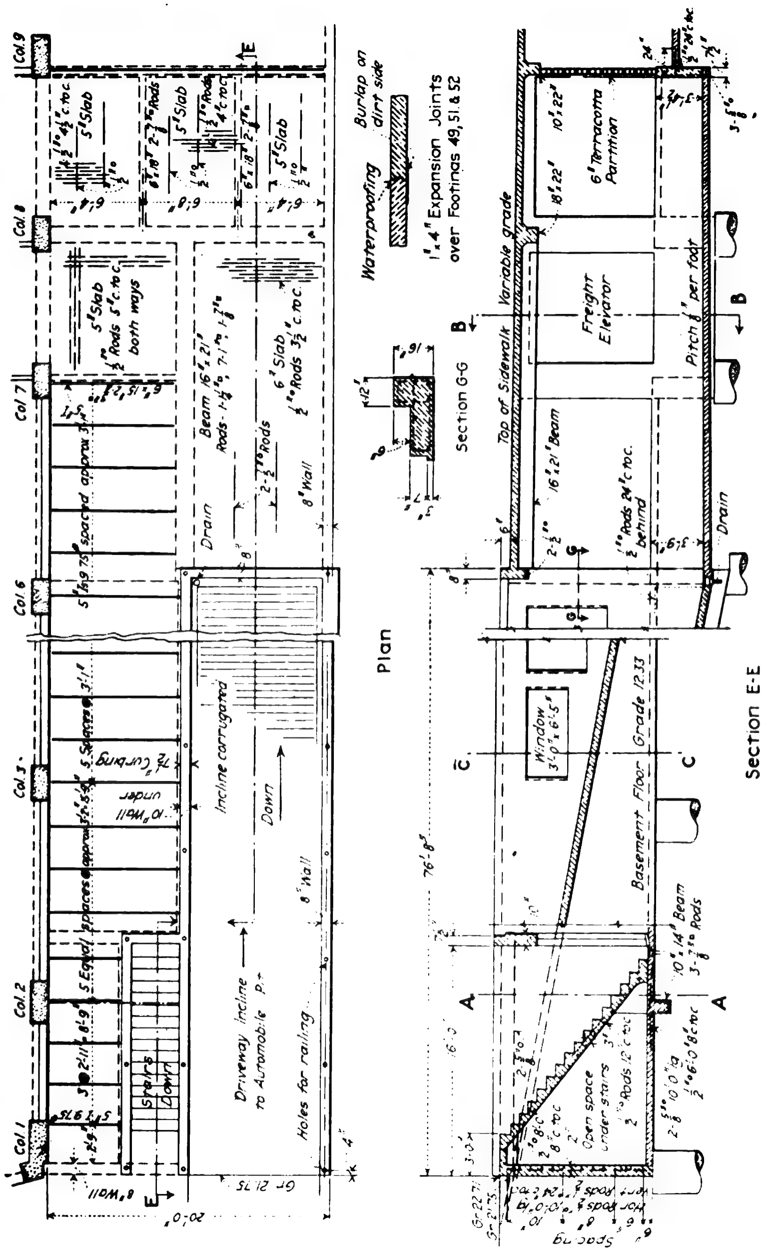


FIG. 235.—Cross Roads building, Boston, Mass.

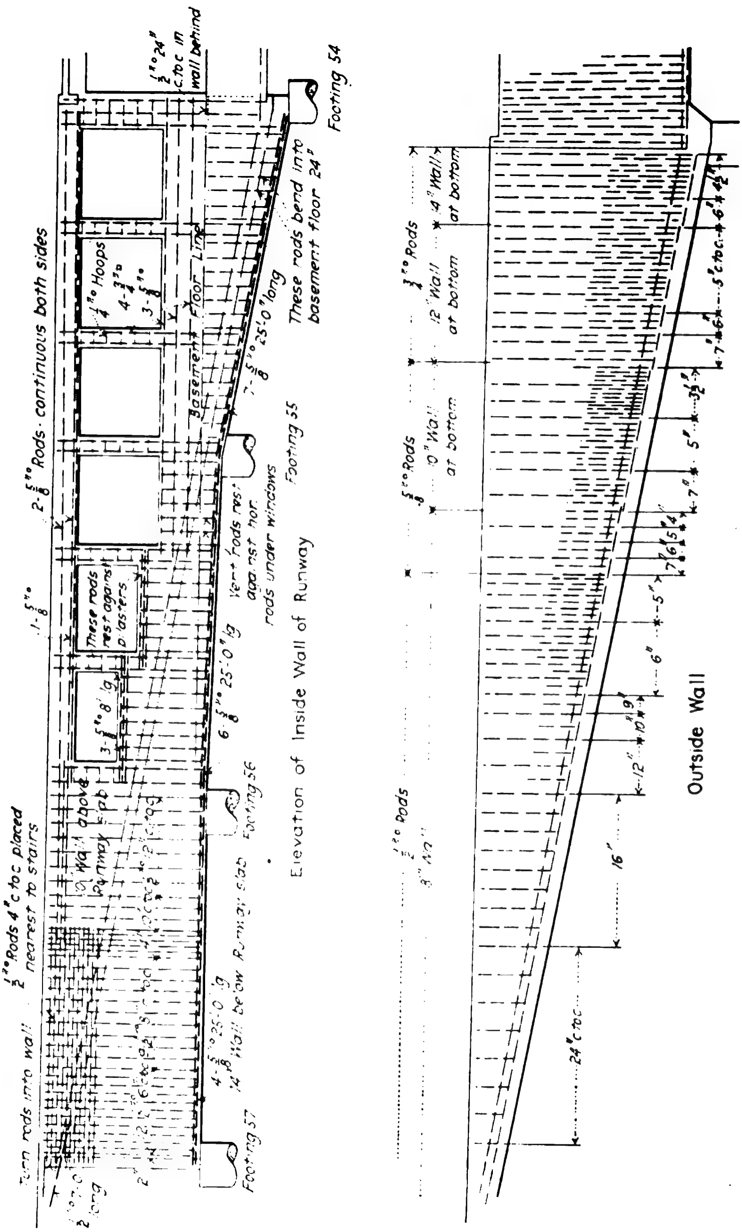
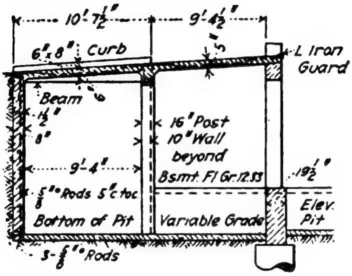
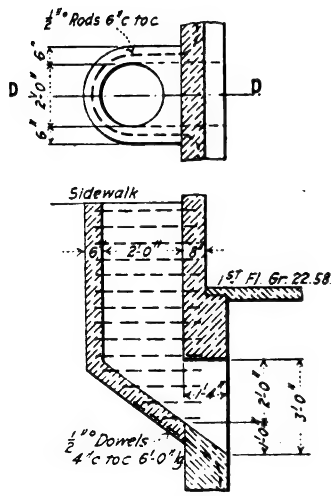


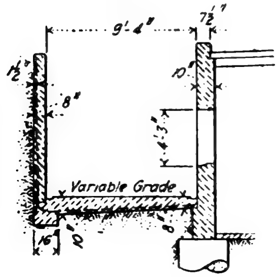
FIG. 236.—Cross Roads building, Boston, Mass.



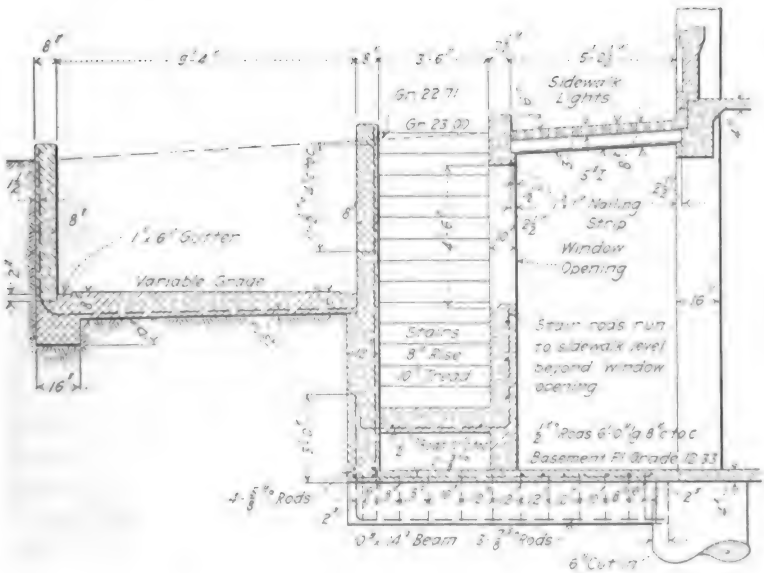
Section B-B



Section D-D
Detail of Coal Chute



Section C-C



Section A-A (See Plan)

Fig. 237.—Cross Roads building. Boston, Mass.

excavated. In order to proceed with the work in this manner, the Concrete Engineering Co., who were the contractors for this building, designed lintel beams along the Beacon St. side to support the first floor temporarily. These lintel beams finally became part of the basement walls, as the details clearly show. The basement wall was made to act as a beam between columns 16 and 18 and to support a column 17 which starts at the first floor level. This was done because it was not considered a feasible proposition to connect column 17 with the next interior column by means of a cantilver footing.

Basement walls under a sidewalk are shown in Figs. 235, 236, and 237. Walls are also shown for a driveway which runs from street level to an automobile repair pit below the basement floor. The student should note that the floor of the driveway and the outside wall form an L-shaped retaining wall and may

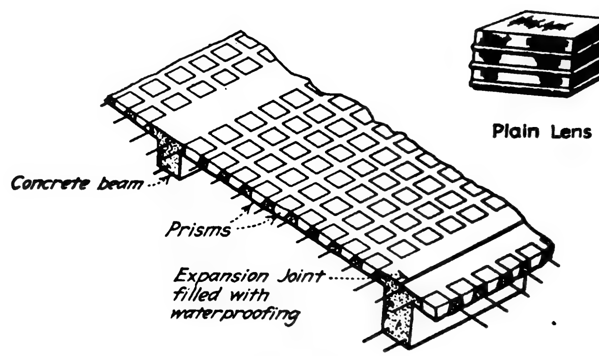


FIG. 238.

be figured by means of the principles explained in Part I of this volume. As the depth of wall increases the thickness at the bottom of the upright portion increases and the steel is varied accordingly. The basement walls proper are held at the top by the sidewalk slab and at the bottom by either the basement floor or the floor of the pit. The expansion joints over the footings should be noted. The details shown are not complete, but it is thought that sufficient data is given to indicate the method of construction.

The sidewalk lights are formed of circular pieces of plate glass set in reinforced-concrete slabs, and supported by steel beams. A typical vault-light construction, using concrete

beams, is shown in Fig. 238. The spacing of the beams should be governed by the thickness of the slab—the usual spacing being 3 to 4 ft.

60. Partitions.—Solid concrete partition walls may be made 3 or 4 in. thick if reinforced in a similar manner to exterior curtain walls. Extra rods should be placed near the edges of all openings, and rods should project into the floor and ceiling for anchorage. It is usually convenient to pour the concrete after the floors are laid, and, where partitions are not located under beams, this may be done by leaving a slot in the floor at the proper place.

Concrete blocks have been used to some extent, but their thickness is often objectionable. In the Wholesale Merchants' warehouse at Nashville, Tenn, concrete blocks were employed 8 by 8 by 24 in. in size with two hollow spaces. The blocks around the elevators were 4 by 4 by 6 in. solid. Rabbets were formed in each end and in top and bottom surfaces, and these were filled with cement mortar as the blocks were laid, in order to secure as perfect a bond as possible.

A solid concrete wall 4 in. thick makes a very efficient fire-resisting partition, but is heavy, and difficult to install. For this reason metal lath and plaster, terra-cotta tile, and plaster-block partitions are generally used in preference to concrete.

Metal lath and plaster partitions are extensively employed in concrete buildings and make a fairly good (although not first-class) fire-resisting construction. Some tests of plaster partitions in actual fires have shown such constructions to be reliable under fairly severe conditions, while other tests have shown failure, due to the difficulty of obtaining sufficient bond between the plaster and the metal lath to resist successfully the combined action of fire and water.

The ordinary form of metal lath and plaster partition consists of channel-iron or other steel studding set crosswise of the partition, and connected at the top and bottom with the floor. The metal lath is then fastened to both sides of the studding and each side plastered with a mixture of lime and cement mortar. Three coats of plaster are generally applied—the first or scratch coat containing the usual quantity of hair or fiber. Openings are cased on all sides with a steel framework to which the lath is firmly fastened. This type of partition is usually made from 4 to 6 in. thick, and made either hollow or with a central body of

cinder concrete. A 2-in. solid partition, however, is sometimes used and consists of only one thickness of metal lath wired to the steel uprights.

Metal lath and plaster partitions made by the Roebling Construction Co. are shown in Figs. 239, 240, and 241. The studs are fastened top and bottom by means of knees. Where flats

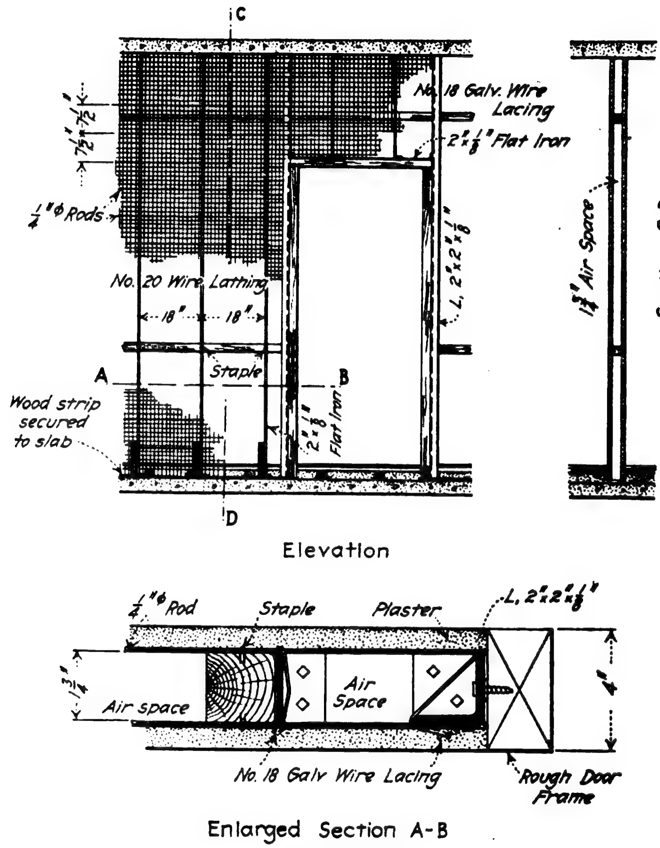


FIG. 239.

are used for uprights, the knees are formed by simply bending the ends of the studs. Where cement-finished floors are used, wood plugs or expansion bolts are inserted for fastening. Wood furrings, where required for nailing purposes, are attached to the studs by means of staples. Furrings are usually required to receive the base-boards, chair rail, and picture molding. They

are not needed, however, where cinder concrete is filled in between the lath since cinder concrete will receive nails readily. The vertical members at door openings are made to extend the full height from floor to ceiling. Such members around openings are punched at intervals with holes to permit the fastening of wood

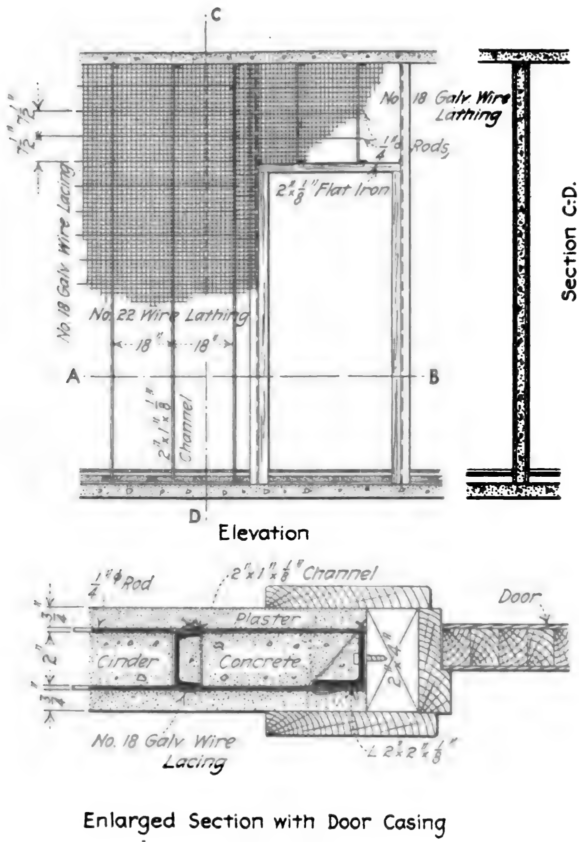


FIG. 240.

frames or other trim. Of course, from a standpoint of fire-resistance, metal or plaster trim is preferable to wood.

Two types of plaster partitions erected by the Expanded Metal Companies are shown in Figs. 242 and 243. A metal and plaster partition known as the *hy-rib* construction is shown in Fig. 244. The ribs of the hy-rib take the place of studs and

eliminate the necessity of attaching sheets of lath to studs. The manner of attaching a hy-rib partition to floor and ceiling is shown. Where hollow partitions are required, two thicknesses of hy-rib are used, separated by means of rib studs.

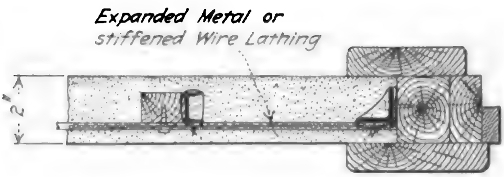


Fig. 241.

Terra-cotta tile partitions are very satisfactory light-weight partitions if made of semi-porous or porous material. Partition blocks are made in thicknesses varying from 2 in. to 12 in., the 4-in. block being the most common. Plaster on both sides of

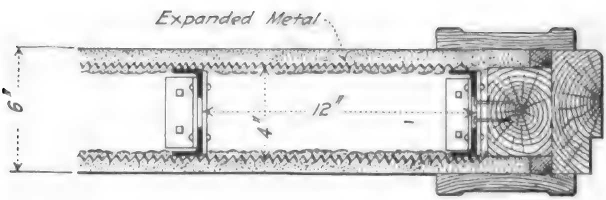


Fig. 242.

a terra-cotta partition increases the thickness by 1½ in., that is, ¾ in. to a side. Two-inch and three-inch tile partitions are not recommended for dependable efficiency in case of fire. The safe height of a terra-cotta partition may be approximated by multi-

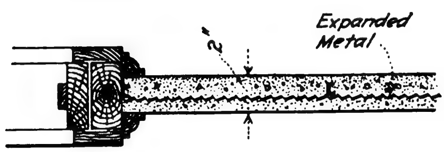


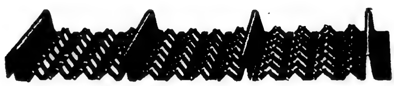
Fig. 243.

plying the thickness of the block in inches by 40. This will give the safe height in inches.

Full-porous terra-cotta blocks are slightly more expensive than semi-porous, as they weigh more per square foot, and have

heavier faces and webs, but they make a partition which is decidedly more dependable under fire test. At least a portion of a partition should be built of full-porous blocks in order to provide for the nailing on of wood trim.

Terra-cotta partitions should be securely braced with slate at the ceiling. The blocks should be wet before being laid and also before being plastered, and should be laid in a mortar composed of one part lime-putty, two parts cement, and two or three parts sand.



Hy-rib

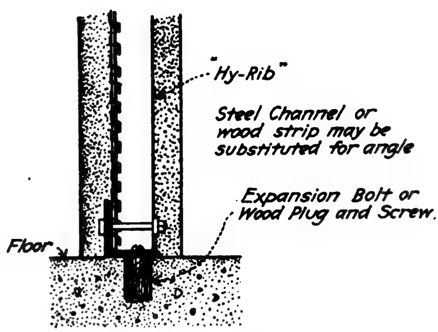


Fig. 244.

Plaster blocks as used in partition construction are made principally of gypsum or plaster of Paris, with an admixture of wood fiber, reeds, or other suitable material. They are extremely light, easy to handle, and can readily be cut and sawed, but possess several disadvantages in actual use. They also offer poor resistance to hose streams in case of fire.

The table on page 308 gives weights of various forms of partitions. The weights for the block partitions do not include the plastered surfaces. If a partition is to be plastered on both sides, add 10 lb. per square foot.

WEIGHTS OF PARTITIONS

Kind of partition	Thickness (inches)	Weight (lb. per sq. ft. of partition)
Solid plaster	2	20
	4	32
Hollow plaster	4	22
Terra-cotta	2	12 to 14
	3	15 to 17
	4	16 to 18
	5	18 to 20
	6	24 to 26
Plaster black	2	7
	4	12
	8	22

Large buildings should be divided by firewalls of either concrete or brick, so that if fire occurs in any part of the structure it will be kept from spreading. A firewall to be thoroughly effective should run from basement to roof and be provided with automatic fireproof doors. Severe exposures require fire doors on each side of such walls.

Stairs and elevator shafts should be enclosed in either concrete or brick partitions. Where considerations of appearance or light prevent the use of opaque enclosures, the fire hazard should be reduced by using wire glass in metal frames. Open grillework around passenger elevators should be of this construction.

PROBLEM

15. Design an 8-in. basement wall, 10 ft. high, to extend between wall columns (16 in. by 24 in.) which are spaced 16 ft. on centers. Consider that the wall is to sustain earth level with the top of the wall and reinforce vertically for the earth pressure. The wall is to have no footing of its own, but will rest on the wall-column footings. Assume the earth pressure as due to a fluid weighing 30 lb. per cubic foot.
16. Design the wall of Problem 15 reinforcing horizontally for the earth pressure.

CHAPTER X

STAIRS

61. General Design.—The usual type of reinforced-concrete stairway consists of an inclined slab with the steps molded upon its upper surface. The design of such stairs is a simple problem,

Courtesy of Ferro-concrete Construction Co.

FIG. 245.—Stairs in MacDonald & Kiley factory, Cincinnati, Ohio.

since the thickness and reinforcement may be calculated as for a slab freely supported at the ends. Transverse reinforcement is used only for stiffening and to prevent shrinkage cracks. A

common load used in designing stairs in commercial and manufacturing buildings is 100 lb. per horizontal square foot.

A simple method of design is to support the ends of a stairway slab directly on floor beams or floor girders, or on some special beam or beams inserted for the purpose. This, however, cannot always be accomplished conveniently and it is common design to make the span of a stairway slab to include a platform slab as well (Fig. 245). Stresses in slabs of this type are somewhat indeterminate on account of the angle which occurs at the edge of the platform; but many such slabs, however, have been computed

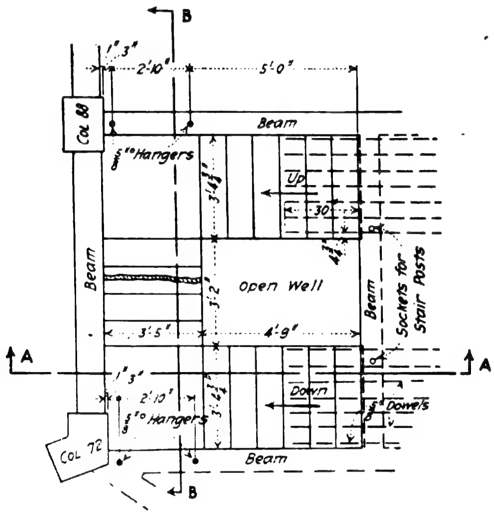


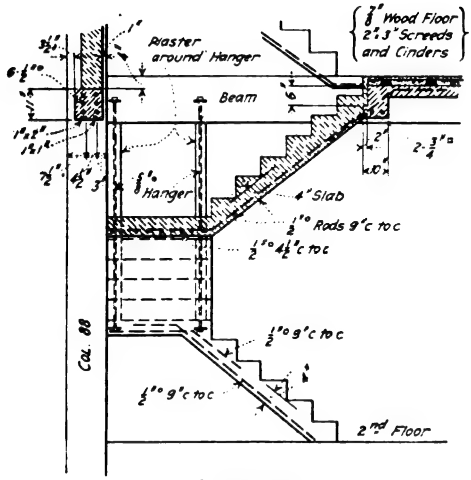
FIG. 246.—Stair details, factory building, Cambridge Building Trust.

as simple slabs freely supported and, as far as the writer knows, have given satisfaction in every case.

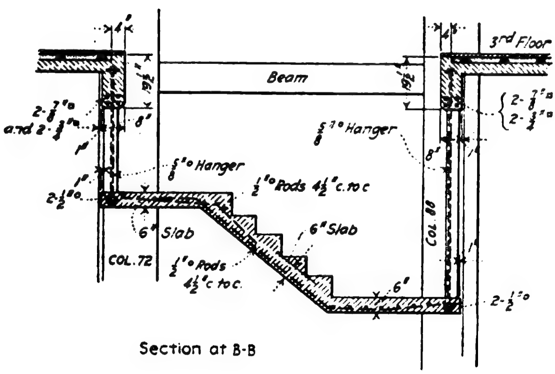
In stairway design, some arrangement is usually made whereby short spans may be employed, as slab construction is not suitable for long flights. When it becomes necessary to use stair spans of great length, however, without any chance of intermediate support, a side girder construction may be employed. The girders may be calculated as simple beams with the reinforcing steel near the lower surface. A small rod is run across from girder to girder at the foot of each riser so that the risers are practically reinforced beams.

Stairways should be enclosed in fire-resisting partitions in order to prevent the spread of fire. (See Art. 60.)

62. Methods of Supporting Stairs.—The slab method of construction is usually the cheaper and employed wherever possible. In long straight flights, a beam is often used to



Section at A-A



Section at B-B

shorten the span. This beam may run between the regular columns which make up the structural frame, or additional short posts may be provided. Fig. 206 shows a stairway supported in this way. The design is that employed near columns 1 and 2 of the Lang Building, Fig. 200.

Where a stairway must be broken into two or more runs to the story, it is customary to employ either rod hangers or beams intermediate in the story height. Figs. 246 and 247 show stairways supported by rod hangers only. In Fig. 248 both intermediate beams and rod hangers are employed.

Courtesy of Universal Portland Cement Co.

Fig. 247.—Stair design. Rogers & Hall Co.'s building, Chicago, Ill.

Stairways are generally constructed at some convenient time after the structural frame is completed. Where stair-runs start from floor beams or floor girders, a plank should be nailed to the side of the beam forms to cause rabbets in the concrete, and dowels should also be provided.

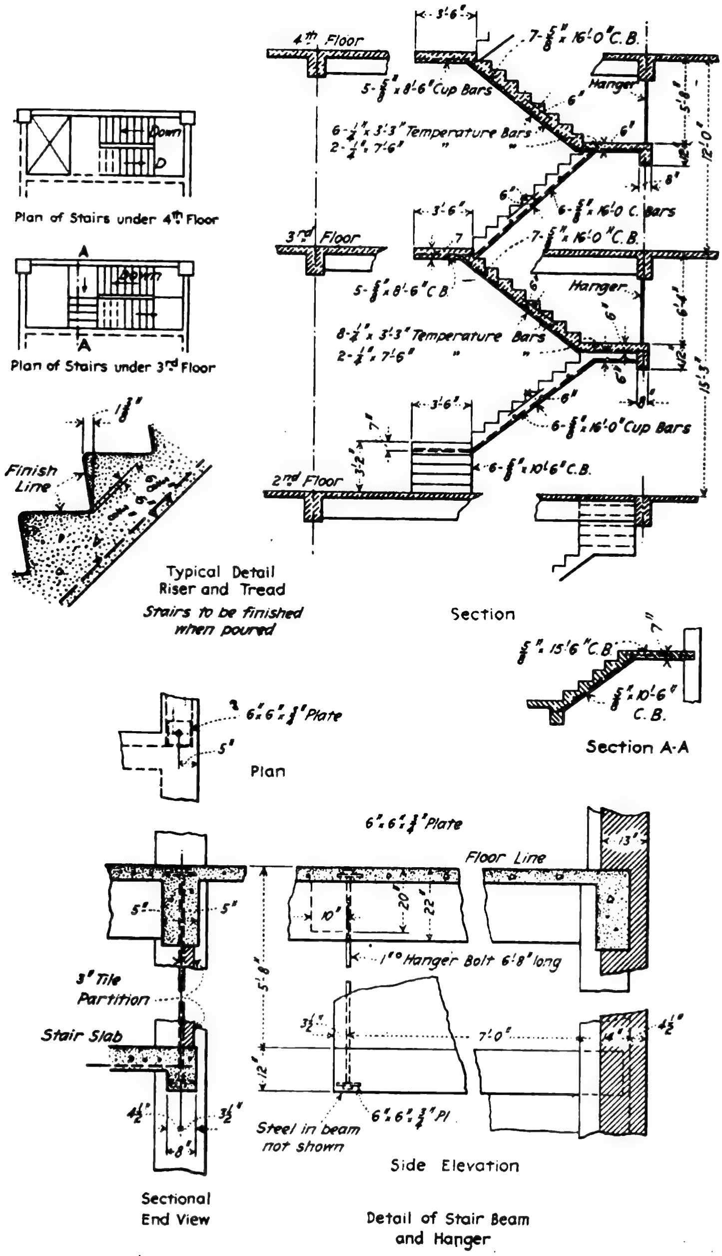


FIG. 248.—Stair details. The Bobbs-Merrill building, Indianapolis, Ind.

The ends of a stairway slab are usually fixed to more or less extent and the dowels provided in the course of construction may well be made of such length and in such numbers as to give a sufficient reinforcement for negative moment. In short spans fixed

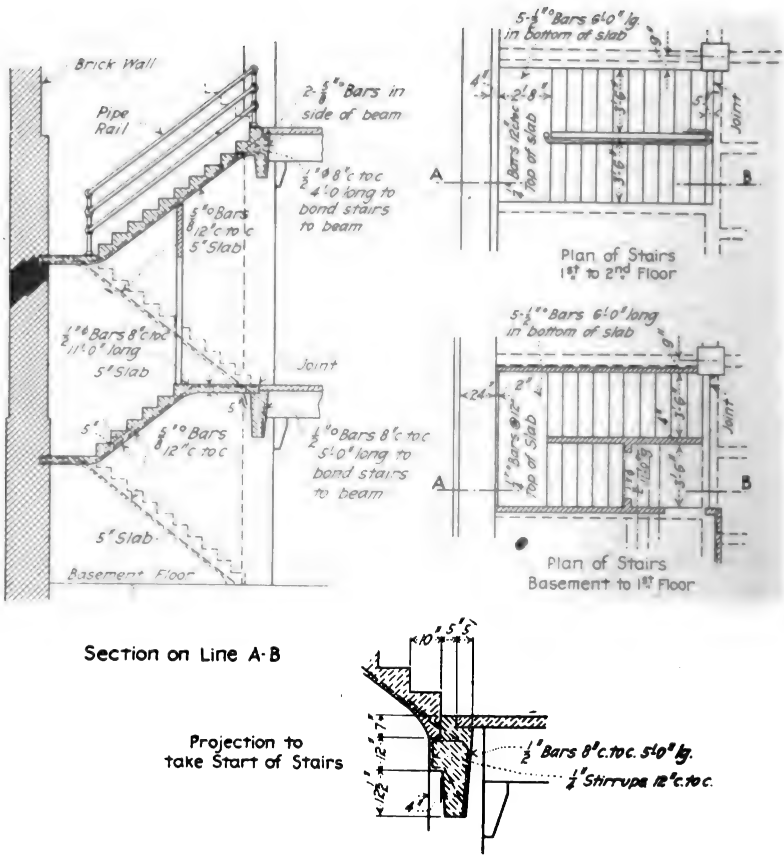
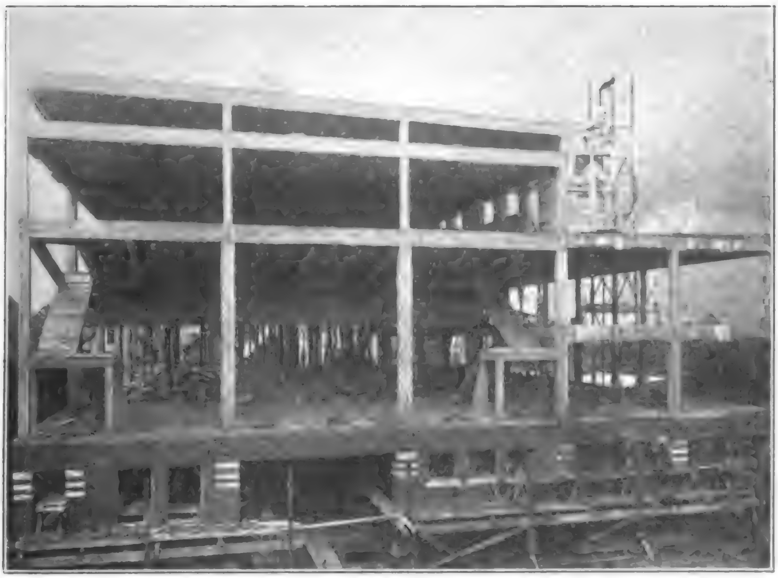


FIG. 249.—Stair details. Warehouse for Portland Savings Bank, Portland, Maine.

in this way, it seems to the writer that the moment at the center of slab may be computed with safety using the formula $M = \frac{wl^2}{10}$

A stairway supported by brick walls is shown in Fig. 249. The same arrangement would be employed with concrete bearing walls.

Steps are sometimes molded after the rough concrete stair slab is in place. The steps may then be molded separately and set on the slab, or they may be poured in place. In the former case, rods should be embedded in each step to permit of handling without injury, while in the latter case the steps are all poured at once in a similar manner to the way the pouring is done when they form an integral part of the slab. Fig. 250 shows this type of stairway in process of construction. The rough slabs only have been poured.



Courtesy of Universal Portland Cement Co.

FIG. 250.—Reinforced-concrete theater building, Milwaukee, Wis.

Fig. 251 shows a bold design in reinforced-concrete stair construction.

63. Stair Details.—In factory construction stair treads are usually surfaced with a 1-in. or $1\frac{1}{2}$-in. cement top finish. A method of constructing the steps is illustrated in Fig. 252. The form board for the riser is arranged as shown at *A* and the top of this board is beveled so that by placing an upper form board on the outside, a nosing or projection in the concrete work is formed.

Another type of step with cement finish is shown in Fig. 248.

This step makes a good appearance, but is a trifle more difficult to construct than the step previously mentioned.

The outer edge of concrete steps are protected in many cases by a light steel angle which runs the entire width of the stairs.

Courtesy of Universal Portland Cement Co.

FIG. 251.—Spiral stairs. Chicago Union Lime Works, Chicago, Ill.

When linoleum is used, this angle is raised so as to have the upper leg flush with the finished tread. Metal non-slipping treads embedded in the concrete are sometimes employed.

Fig. 246 shows the method of connecting concrete stairs with a wood-finished floor. Fig. 253 gives the details of wood-

covered concrete stairs employed in a fireproof residence. In the best buildings, treads and risers are often covered with marble slabs having plaster soffits.

The *rise* of a stair is the height from the top of one step to the top of the next. The *run* is the horizontal distance from the face of one riser to the face of the next. The *run* is usually less than the width of tread on account of the nosing. To secure a

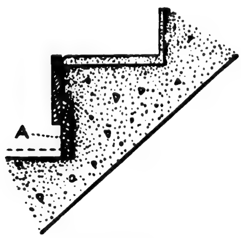


Fig. 252.

comfortable stair, the run must bear a certain relation to the rise. One rule is that the sum of the rise and run should be equal to from 17 to 17½ in. For ordinary use, a rise of 7 to 7½ in. is about right. Stairs having a rise greater than 7¾ in. are steep. Properly designed stairs without nosings should have at least 12-in. treads to be comfortable and the above rule does not apply to this type of stair.

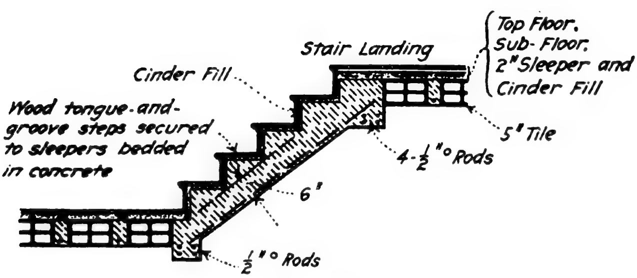


Fig. 253.

The common railing is of galvanized-iron pipe put together with malleable-iron fittings and having the stanchions rigidly secured in sockets in the concrete work. Fig. 207 gives details of a railing of this type used in the Lang Building. The handrail should be placed at a height of about 2 ft. 6 in. above the

tread on line with the face of riser. Rails monolithic with the stairs are sometimes used.

Problem

17. Assume the floor design of Problem 5. Use a part of a floor bay in the interior of the building for a stair shaft and provide enclosure partitions. Take the height from finished floor to finished floor at 15 ft. and design the stairs through one story using three runs. Submit computations and complete design.

CHAPTER XI

ELEVATOR SHAFTS

64. Elevators in General.—In order to appreciate the detailed design of elevator shafts, it is necessary to have a general idea of the different types of elevators and the chief points concerning their construction.

As the matter stands at the present day, elevators installed are either *electric* or *hydraulic*. The hydraulic is the older of the two main types, but the electric elevator possesses several advantages over the hydraulic type and seems to be more universally used.

Electric Elevators

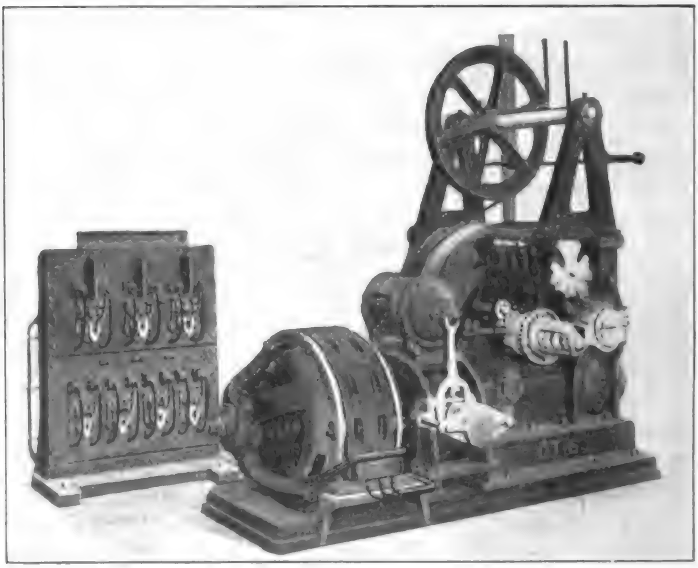
Electric elevators are of either the *drum* or traction type. The machine for the drum type (Fig. 254) consists of a worm and gear wheel actuated by an electric motor, the gear wheel being attached to a winding drum or spool. Four cables wind upon the drum, two of which lead to the car and two to the machine counterweight. The drum type is not suitable for buildings of great height because the drum length required would make a machine too large to be installed under ordinary conditions. In many cases, also, the deflection of the cables would be greater than could be accommodated in the size of the elevator shaft. Then, too, the development of the drum type of apparatus has not been such that suitable speed for any great amount of travel could be obtained.

A typical installation of the drum type with the machine located in the basement is shown in Fig. 255. In order for the student to appreciate the arrangement of the cables, the method of what is called *counterbalancing* should be understood.

The simplest way to counterbalance a car is to attach a cable (other than the lifting cable) to the top of the car frame, leading this cable over one or more overhead sheaves (Fig. 256) and suspending the counterweight therefrom. In order that the car may descend when empty, the counterweight must, when so attached,

always be less than the weight of the empty car with its fixtures. No power is needed for the down trip of the car, but on the up trip, the motor must develop enough power to raise the maximum load plus the unbalanced weight of the car.

In all drum elevators, however, the motor and drum are reversible and power can be applied during the down trip as well as during the up trip. Under these conditions it becomes possible to counterbalance, not only for the full weight of the car, but also for the average load. This may be accomplished by



Courtesy of Otis Elevator Co.

FIG. 254.

attaching the counterweight to the drum and winding the cable in an opposite direction to that of the lifting cable (Fig. 257), so that when the lifting cables are unwound, the counterweight cables are wound on the drum and vice versa—thus allowing the lifting and counterweight cables to travel in the same groove. The advantage of this method of counterbalancing (called *overbalancing*) compared with the first method is apparent. If the load on the car is equal to the average load, no power is needed besides that necessary to start the machinery and keep it moving against frictional resistances. If the load is equal to the maximum load and the car is going up, the motor must furnish power

sufficient to raise the difference between the maximum and the average load. If the car is empty and going down, the other

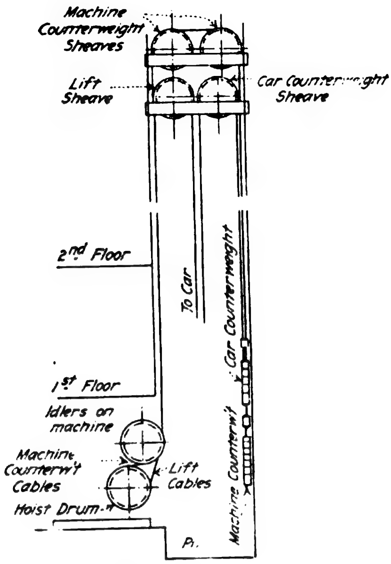


FIG. 255.

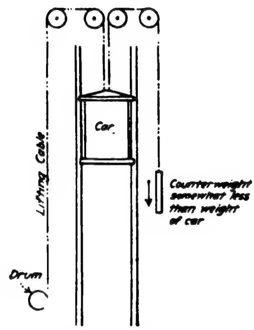


FIG. 256.

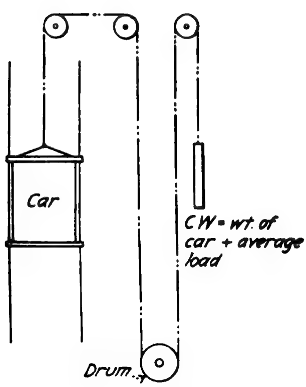


FIG. 257.

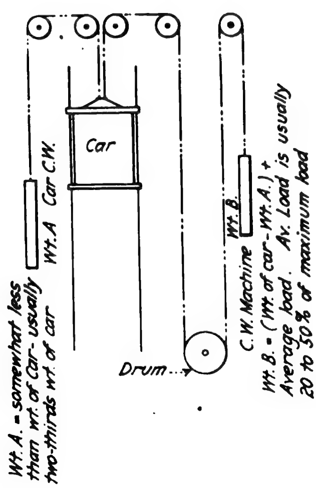
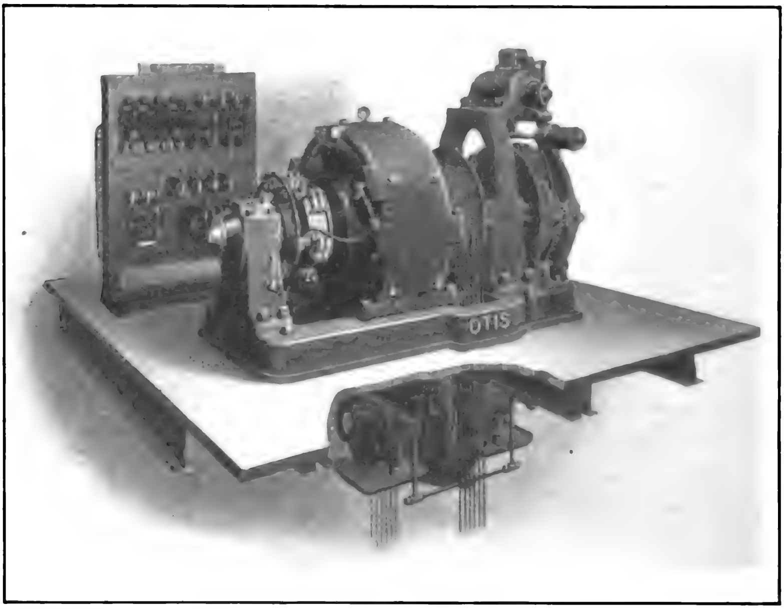


FIG. 258.

extreme, the motor must raise the counterweight with a force equal to the weight of the average load.

Fig. 258 shows a scheme of overbalancing by which the stress in the cables and the pressure on the drum-shaft bearings may be diminished. This scheme is essentially that shown in Fig. 255 with the exception of the arrangement of the sheaves. The number and location of the sheaves, however, do not in any way change the principle involved. Two counterweights are shown, one attached directly to the car and the other to the drum. The car counterweight must be less than the weight of car in order

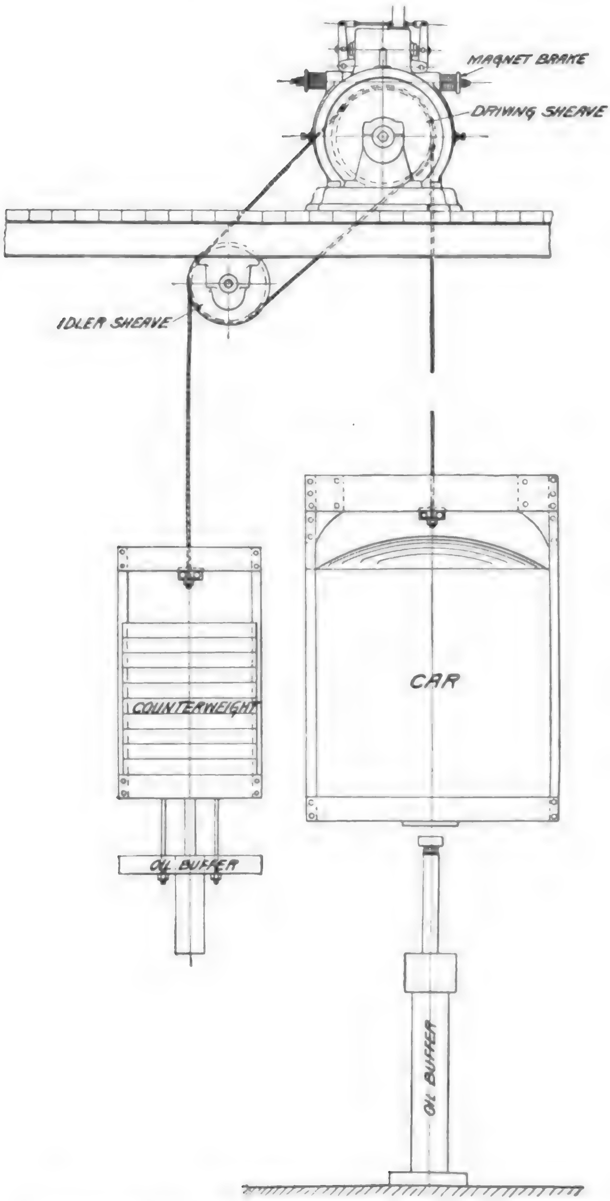


Courtesy of Otis Elevator Co.

FIG. 259.—Elevator machine of the gearless traction type.

to allow the car to descend when empty. The other counterweight is equal to the remainder of the car weight plus the average load.

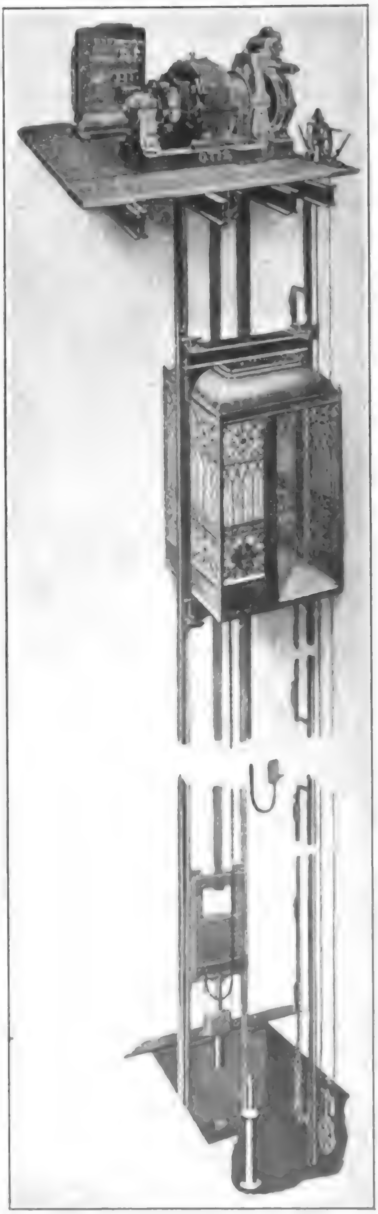
The weight of the cables is a considerable item, especially for high lifts. A varying counterweight, usually in the form of a chain composed of heavy links, is provided for the purpose of balancing the varying weight of the cables on the two sides of the overhead sheaves and thus to counteract any variation in power requirements. One end of the chain is fastened to the bottom of the car, while the other end is made fast to the bottom



Courtesy of Otis Elevator Co.

Fig. 260.—General arrangement of roping for traction elevator installation.

of the regular counterweight. As the car ascends, more and more of the weight of the chain is transferred from the counterweight to the car, and as the car descends the reverse occurs.

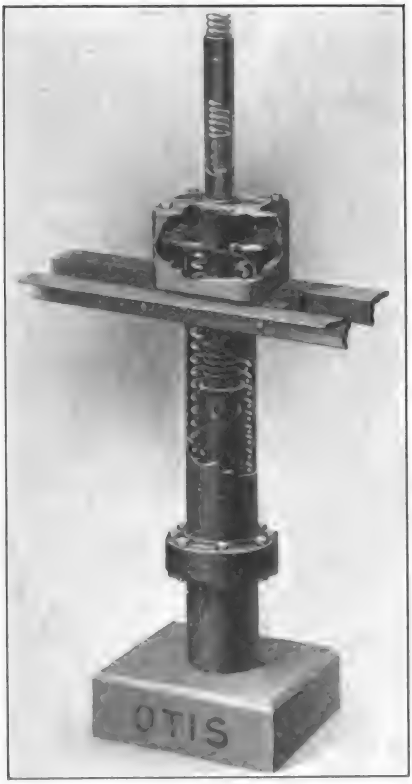


Courtesy of Otis Elevator Co.
FIG. 261.

In the drum type of electric elevator, the machine must be provided with limit switches which cut off the power from the motor and apply the brake when the car reaches the top or bottom of the shaft. There is always danger of the motor continuing to revolve after the car has reached the end of the up-travel. For this reason a sufficient over-run should be provided between the top of the car frame (when the car platform is flush with the top landing) and the lowest point of the overhead work. It may be valuable to note here that most cities of any size and some states have passed ordinances governing the minimum overhead work, and these requirements are usually in excess of what has heretofore been considered fairly good practice.

The traction type of electric elevator (Figs. 259 and 260) was brought forward some eight or nine years ago, designed with the idea of eliminating the drum winding and thus removing the limitation upon the height of travel which the drum winding imposed. This type of elevator now promises to become the standard for high-

rise, high-speed work. The operating mechanism of this elevator is extremely simple, consisting of a slow-speed motor direct-connected to a grooved wheel or sheave. Vertically above or below this wheel, depending upon whether the motor is located at the bottom or top of the shaft, is placed an idler. The cables

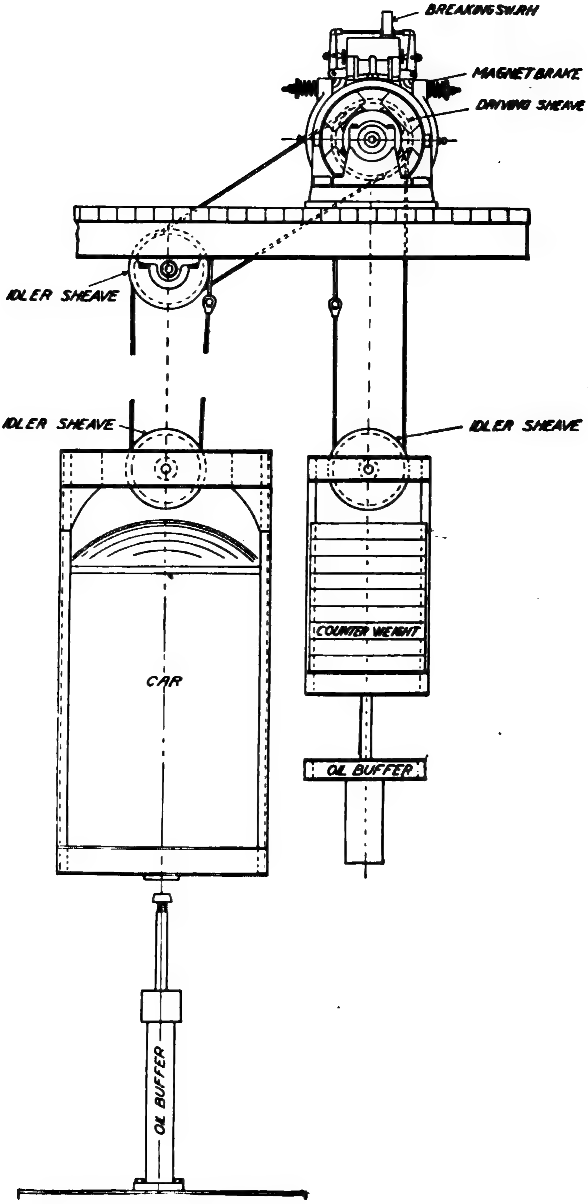


Courtesy of Otis Elevator Co.

FIG. 262.—Otis patented spring return oil buffer.

connecting the car and the counterweight are wrapped around this grooved wheel, as shown in Fig. 261 which is an overhead installation. The weight of the car and the counterbalance produce sufficient adhesion between the cables and grooved wheel to cause the cables to move at the same speed as the circumference of the wheel.

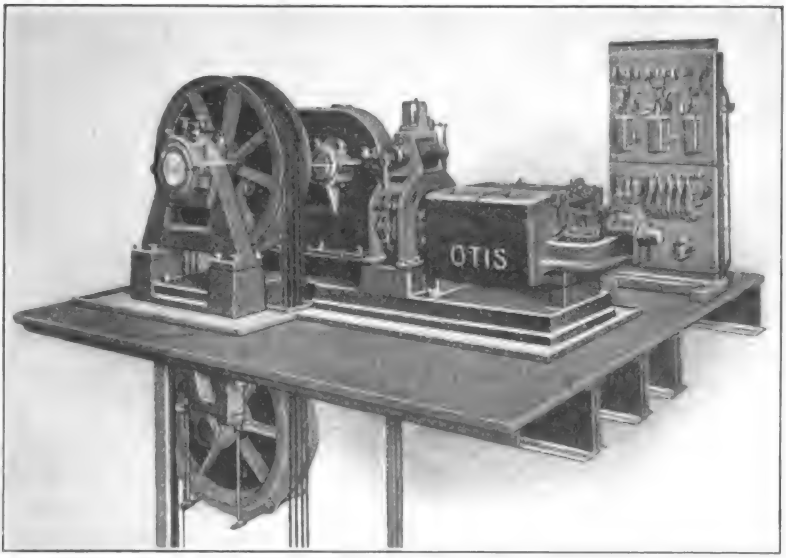
One of the striking advantages resulting from this arrangement



Courtesy of Otis Elevator Co.

Fig. 263.—General arrangement of roping for 2 : 1 traction elevator installation.

of cables and the method of driving the same is the decrease in traction which occurs when either the car or the counterweight strikes the bottom of the shaft. Suppose the motor should continue to revolve after the car has reached the end of the up-travel. The counterweight would then be at the bottom of the shaft and the cables would slacken, allowing the motor to revolve without causing any further movement of the car. The same conditions would result from the car reaching the lower end of its travel. The decrease in tractive effort is a valuable and effective safety feature inherent in this type of elevator.



Courtesy of Otis Elevator Co.

FIG. 264.

Since the traction type is usually selected where the speed and rise are high, the usual safety devices installed in connection with modern high-grade apparatus are used with this type of elevator. An automatic stop device is usually the principal safety feature and is designed to be entirely independent of the brake that is operated in the car. This device is so designed in connection with the motor that the car is initially retarded and finally brought to a positive stop at the upper and lower terminals of travel. The car operating device may be left in the full-speed position and still the automatic device would be effective.

Other safety devices include speed governors; wedge clamps for gripping the rails in case of the car attaining excessive speed; and oil-cushion buffers (Fig. 262) placed at the bottom of the shaft, one under the car and one under the counterweight. The rate of speed at which either the car or counterweight is brought to a positive stop, through the telescoping of the buffer, is regulated by the escape of oil from one chamber of the buffer to another. The buffers have proved capable of bringing a loaded car safely to rest from full speed.

Fig. 263 shows the general arrangement of roping for what is called a 2:1 traction elevator installation—the latest modification in traction apparatus. By the arrangement shown, a tackle is formed which permits of slower car speeds, and at the same time permits the use of a motor of somewhat higher speed than in the ordinary traction type.

The worm-gear or geared traction machine (Fig. 264) grew out of the drum type of apparatus. It permits the use of a motor of standard speed, thereby resulting in a somewhat higher motor efficiency and a lower motor cost. This machine is similar in principle to the direct traction type with the exception that the worm-gearing is interposed between the motor shaft and the shaft carrying the grooved wheel over which the cables pass.

Hydraulic Elevators

Hydraulic elevators are of either the *vertical-cylinder*, *horizontal-cylinder*, or *plunger* types. The vertical-cylinder is perhaps the most common form of hydraulic elevator although the plunger type several years ago enjoyed a considerable degree of popularity for high-rise work. As far as new installations for this class of work are concerned, the plunger type has been practically superseded by the traction type of electric elevator. The horizontal-cylinder elevator is but little used.

In the vertical-cylinder type of hydraulic elevator, the cylinder is usually located along one side of the elevator shaft, but sometimes it is placed in a separate shaft nearby. The cylinder contains a piston which is connected by rods to a traveling frame in which sheaves are mounted. Water under pressure, admitted by means of suitable valves, causes the piston to move from one end of the cylinder to the other. As shown in Fig. 265, fixed sheaves are located at a sufficient height above the traveling

sheaves to accommodate the piston stroke, and the cables which are attached to the car pass over sheaves at the top of the elevator shaft, then down around the traveling sheaves, then up and around the fixed sheaves, down again to the traveling sheaves and then up to the fixed sheaves where their ends are anchored. By this arrangement of the cables, a tackle is formed which multiplies the stroke of the piston. The cylinder itself is usually limited in height to approximately 30 ft. but, by means of the tackle, or *gear* as it is called, this type of elevator has been employed for exceedingly high rises.

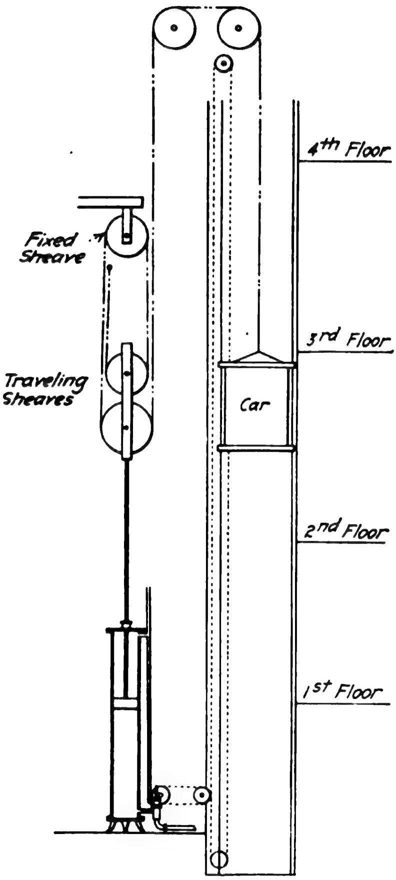


FIG. 265.

The horizontal-cylinder elevator is similar in principle to the vertical type and is

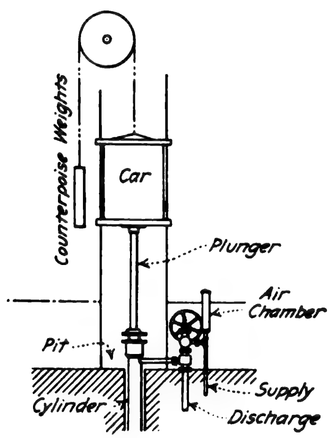


FIG. 266.

found in two forms known as the *pushing* and *pulling* types. In order to save space, the gear of these machines is usually considerably higher than that commonly found in the vertical type machines. The plunger elevator is represented sufficiently in Fig. 266 and needs no description.

All elevators of the hydraulic type depend upon gravity to

effect retardation on the up-trip and to cause the car to descend. The method of counterbalancing in every case is similar to that shown in Fig. 256. The counterweight in the vertical type is provided in part by the weight of the moving parts.

The valves of hydraulic slow-speed freight elevators are commonly operated by a hand rope (Fig. 265) passing through the car and over small sheaves at the top and bottom of the shaft, and connected with the main valve in the basement. Running ropes and standing ropes, either of which may be operated by means of a lever or wheel in the car, are generally used for manipulating the water valves of high-speed elevators.

Compensating chains are used for the geared type of hydraulic machines in the same manner as explained for electric elevators. In connection with plunger elevators, the counterweights themselves are attached to the car frame by cables of sufficient size and weight to compensate for the difference in head as the plunger travels through the cylinder—that is, they make up for the weight displaced.

Selection of Elevator Type

The very large majority of elevators installed at the present day are of the electric type for both freight and passenger service. Hydraulic elevators are occasionally used where conditions seem to make them desirable, but, unless the installation is to consist of a large enough number of elevators to warrant the installation of a good-sized pumping plant, the cost of operating an hydraulic elevator is considerably more than the cost of operating an electric elevator, and would require also the installation of a boiler plant in the building, which is not always desirable. In hotels, hospitals, and in some office buildings where a power plant for lighting and heating is essential, hydraulic elevators are sometimes used, and, if there are enough of them, they can be run probably with as great economy as electric elevators. Under other conditions, however, the electric elevator has almost entirely supplanted the hydraulic.

Examples of Electric-elevator Installations

Figs. 267 and 268 indicate the usual arrangements for ordinary freight elevators, with speeds up to about 150 ft. per minute,

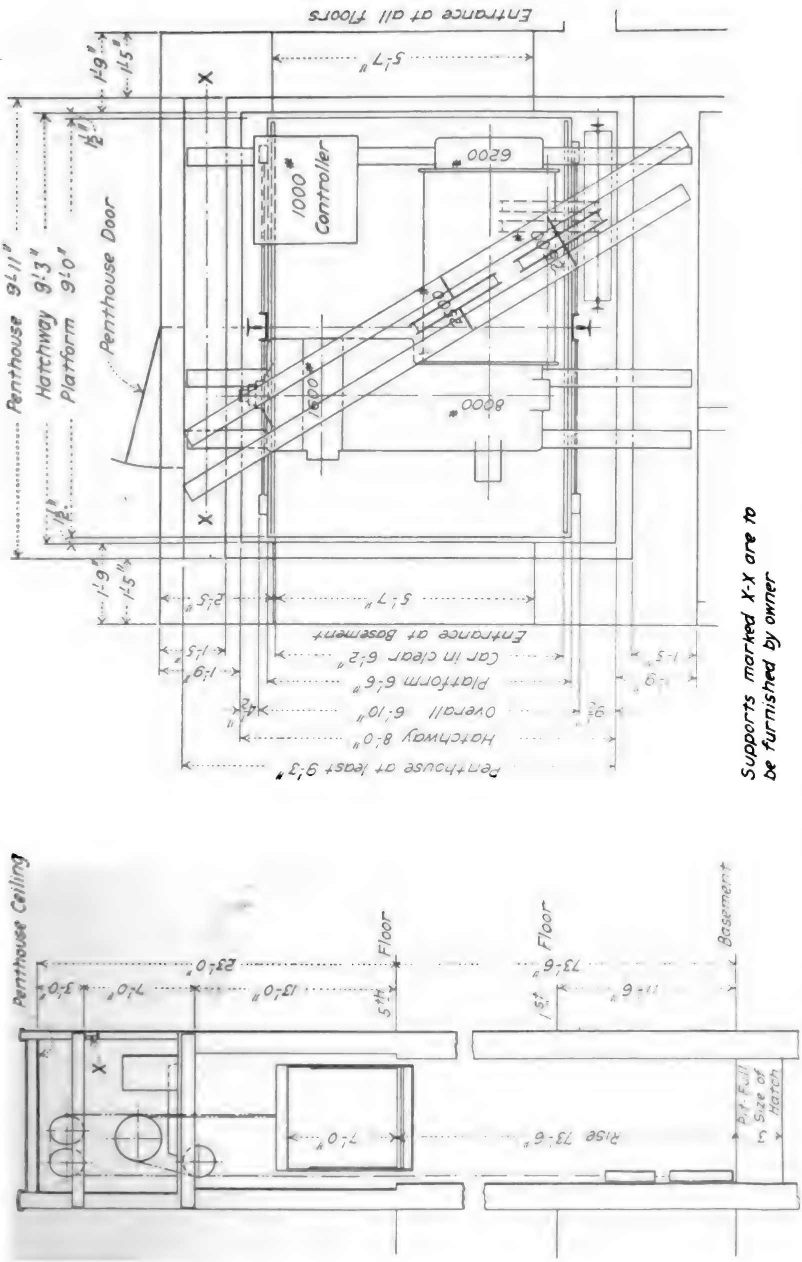


FIG. 267.—One electric freight elevator. Maximum load, 3300 lb. Speed, 3300 lb. at 125 ft. per minute. Drum diameter, 34 in. Face, 49 in.

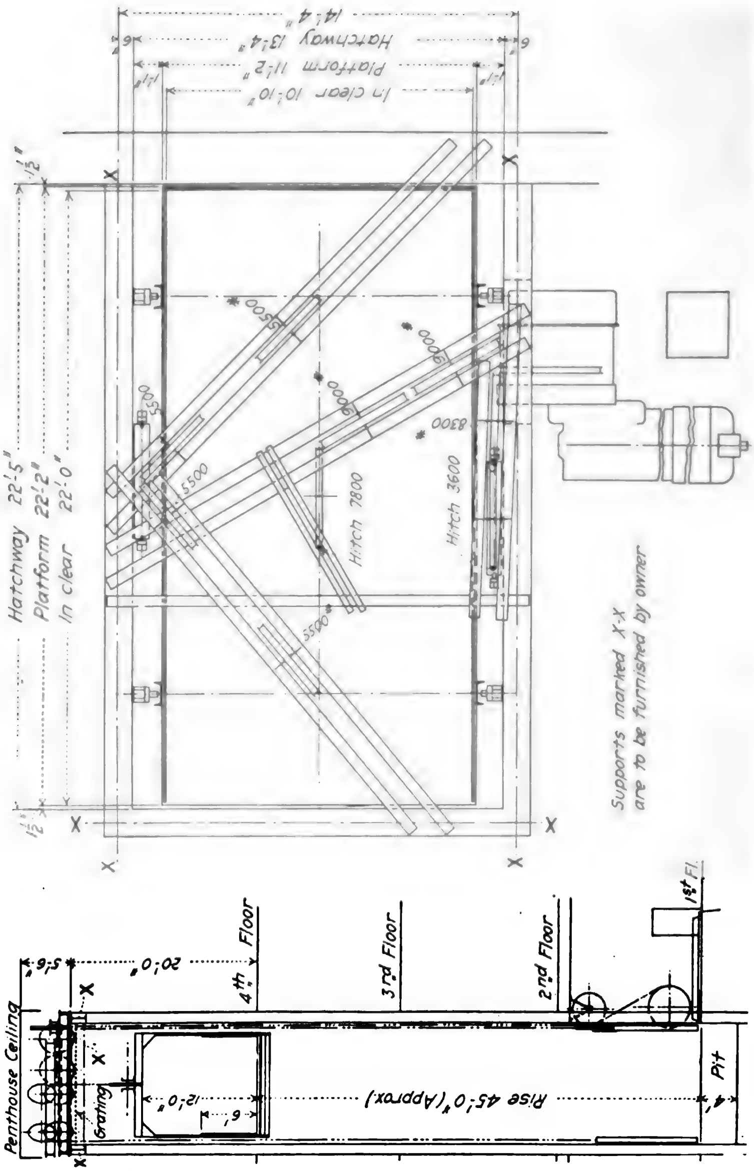


FIG. 288.—One electric freight elevator. Maximum load, 14,000 lb. Speed, 14,000 lb. at 50 ft. per minute. Drum diameter, 48 in. Face, 25 in.

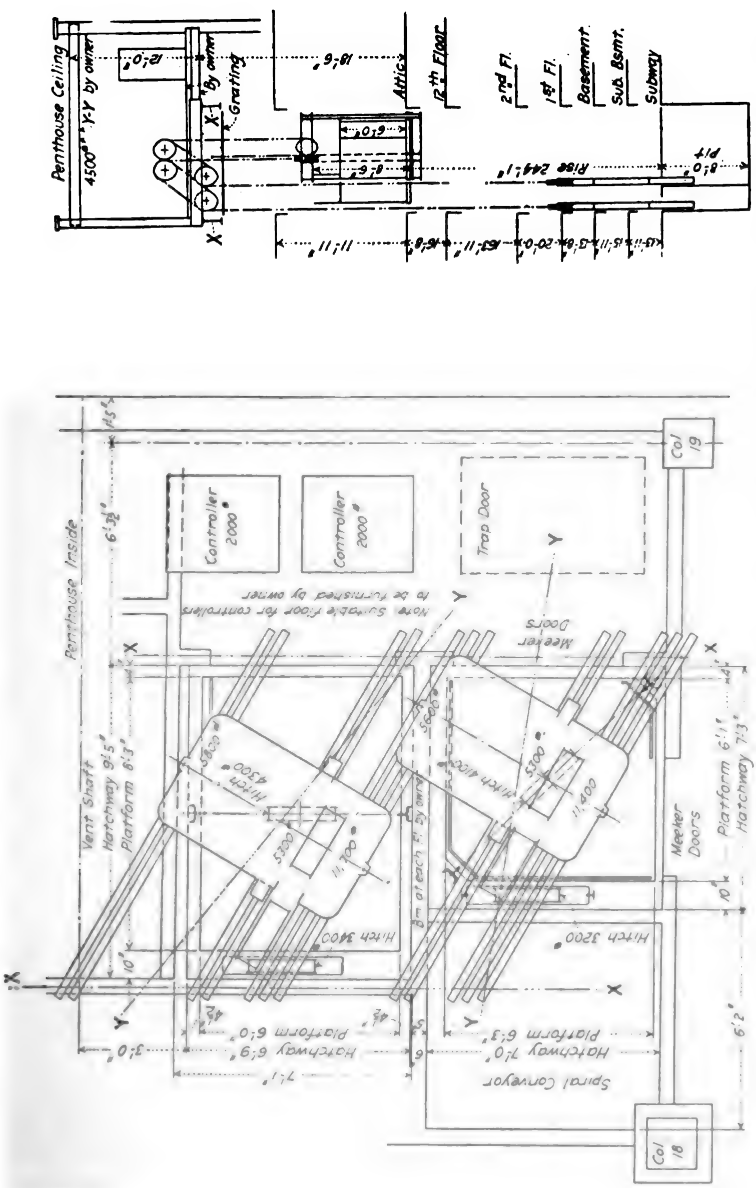


FIG. 269.—Two electric freight elevators. Maximum load, 3000 lb. Speed 3000 lb. at 400 ft. per minute.

where the usual type of drum machines are used. In Fig. 267 the electric machine is located over the shaft, and in Fig. 268 the machine is located in the basement or first floor.

It should be noted that the loads are indicated where they occur in connection with the overhead work. The beams are

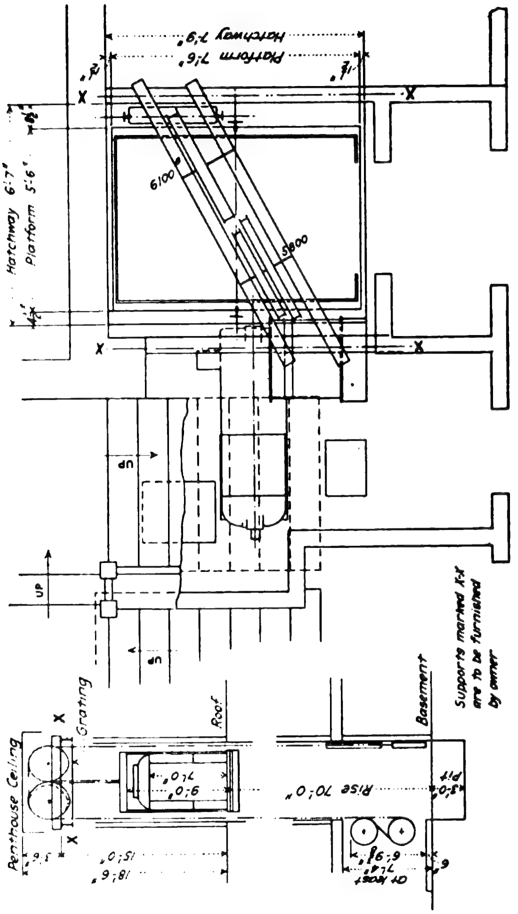


FIG. 270.—One electric passenger elevator. Maximum load, 1200 lb. Speed, 1000 lb. at 100 ft. per minute. Drum diameter, 28 in. Face, 25 in.

also indicated for carrying the sheaves or machines, and these beams transmit the loads to the structural frame of the building. These loads are all actual loads and should be doubled on account of the shock of stopping and starting the elevator car.

Fig. 269 represents freight elevators where the speed is con-

siderably higher, the machines being of the traction type with 2:1 roping, as shown in Fig. 263. This machine is much heavier than drum machines, for the same load to be lifted. This type

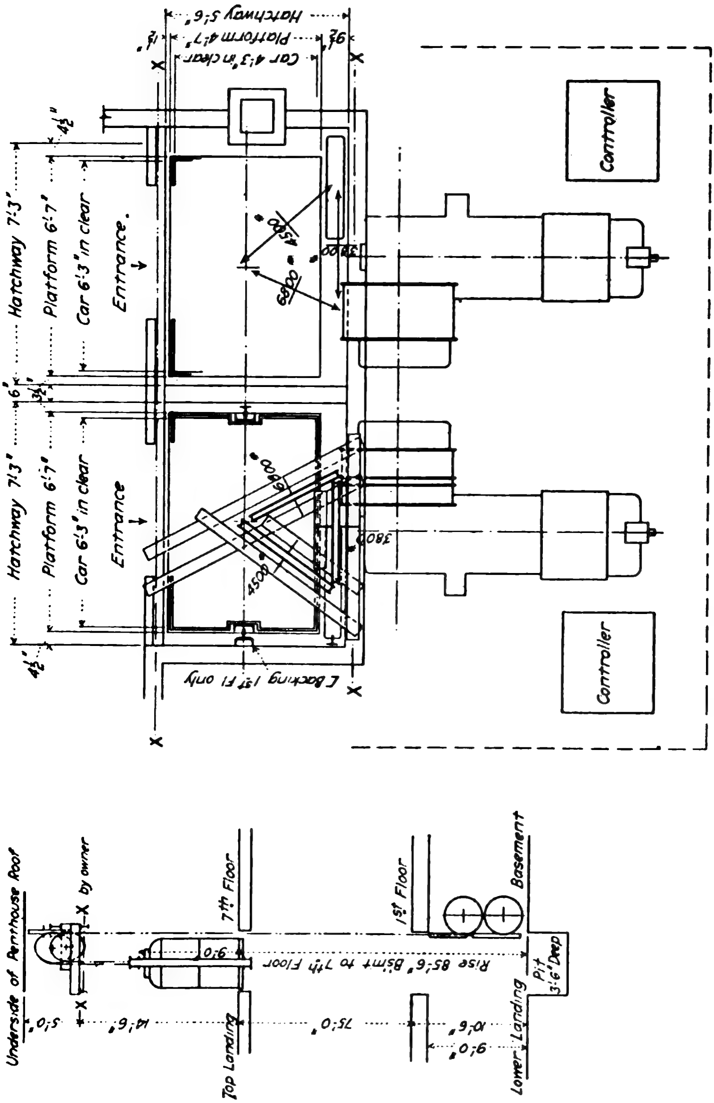


FIG. 271.—Two electric passenger elevators. Maximum load, 2500 lb. Speed, 2000 lb. at 300 ft. per minute. Drum diameter, 40 in.

of installation is employed where the travel of the elevator is considerable, as in office buildings and large warehouses.

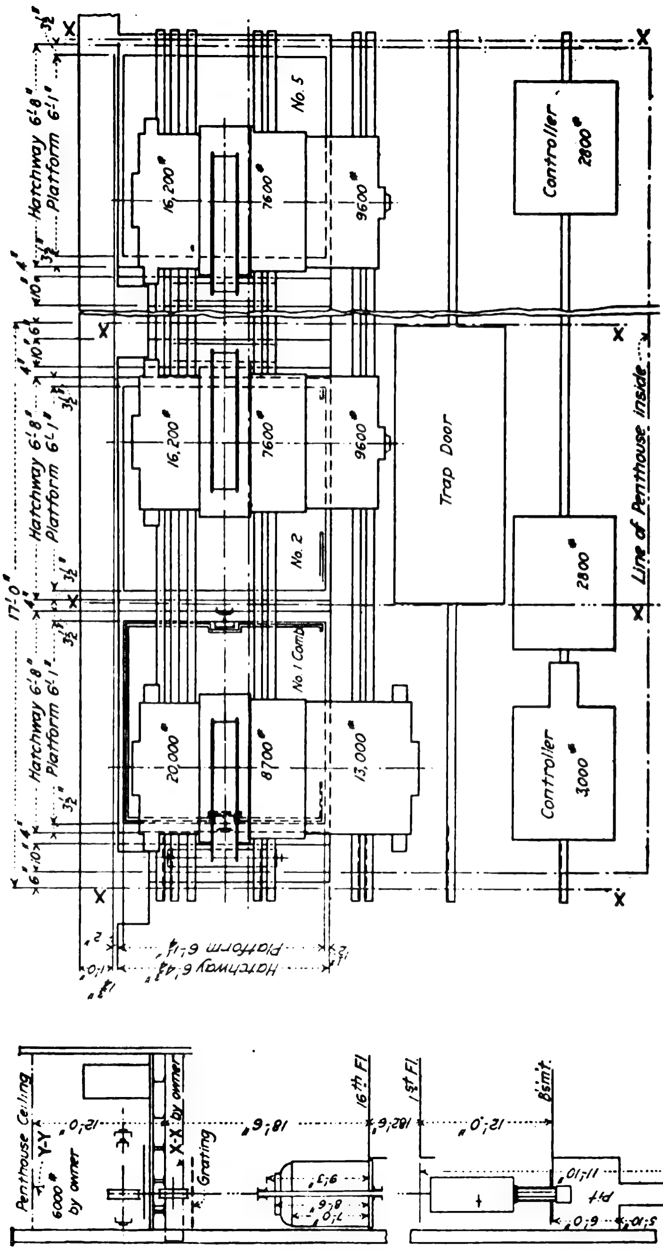


FIG. 272.—Four electric passenger elevators and one combination elevator. Maximum load, 2750 lb. Speed, 2750 lb. at 500 to 550 ft. per minute.

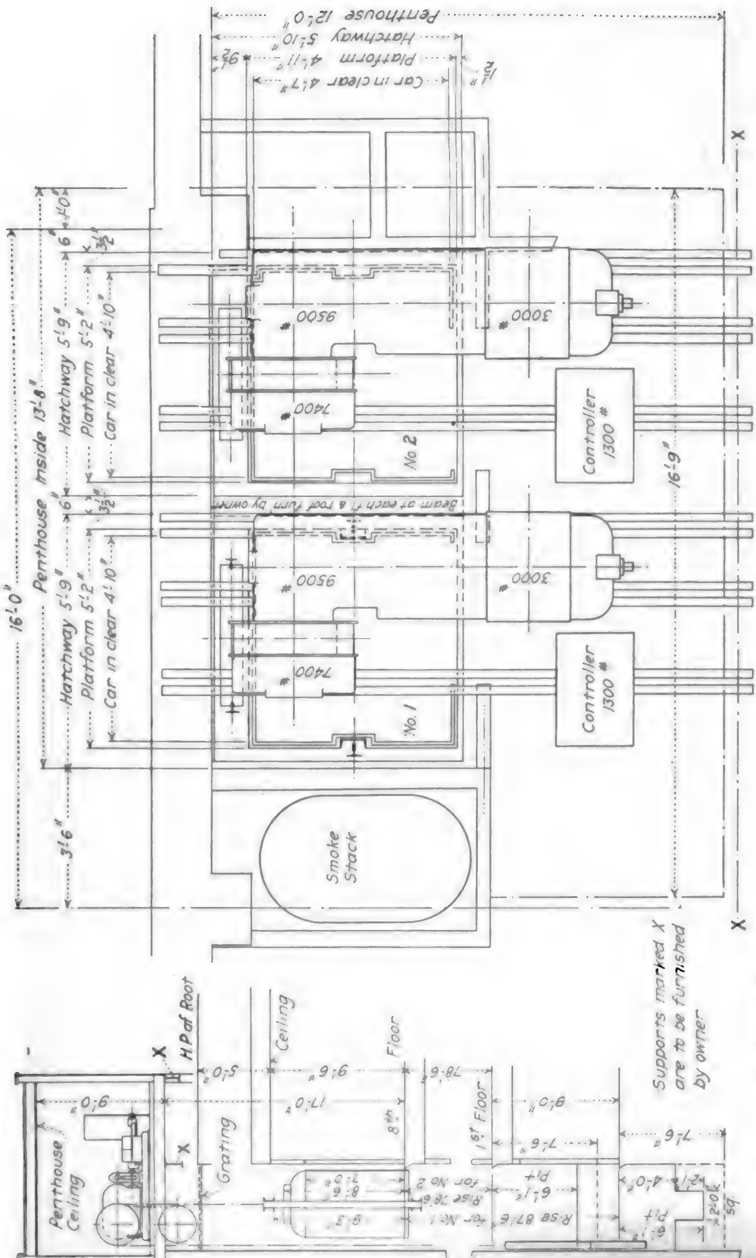


FIG. 273.—Two electric passenger elevators. Maximum load, 2000 lb. Speed, 1600 lb. at 250 ft. per minute.

Figs. 270 and 271 show passenger elevators with drum machines located in the basement, and Fig. 270 shows a frequent arrangement of overhead work which is desirable on account of the small amount of overhead height which it occupies. The arrangement shown in Fig. 271 is mechanically more desirable but requires more height, which makes it less acceptable to architects in general.

Fig. 272 shows passenger elevators of the gearless traction type of machine with 1:1 roping, as shown in Fig. 260. The loads for this machine are very heavy compared to those shown with other types. This is due to the fact that the motor of this machine is exceedingly slow speed, consequently all parts are rather heavy. This machine is used for high rises and confined generally to office buildings and large hotels.

Fig. 273 represents passenger elevators with the geared traction type of machine, similar to that illustrated in Fig. 264. This machine is considered one step in advance of the drum machine and is used for higher rises and usually higher speeds than the drum machines. The loads are approximately the same, but where machines are placed in the basement, which is rarely done, the loads are somewhat greater than for machines of the drum type.

The drawings shown are somewhat conventional and sketchy. They are proposal layouts put out by the elevator contractor to indicate to the architect or engineer what is required in the way of preparation at the building for the accommodation of the elevator apparatus. Architects and engineers preparing building plans should consult with elevator contractors as early as possible in order that each situation may be considered, and in order that proposal drawings similar to those shown above may be obtained before the building is planned in detail. When it is the intention to install either the plunger or geared hydraulic type of elevator, the manufacturer should be consulted without delay, as to the space required for the machinery, boilers, tanks, cylinders, etc., in order to provide ample room for them on the plans.

65. Elevator-shaft Pits.—Elevator-shaft pits should not be less than 3 ft. below the basement floor. When oil-cushion buffers are employed, however, the pit depth should vary according to the speed of the elevator and the stroke of the buffer designed for that particular speed. With a car speed of 600 ft.

per minute, the pit should be about 11 ft. deep at the center where the buffer is placed.

If the first floor is made the bottom landing, a pit pan may be suspended from the first floor, and buffers, if used, may be placed on the basement floor and arranged to project up into the pit

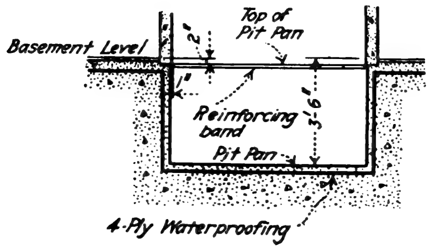


FIG. 274.

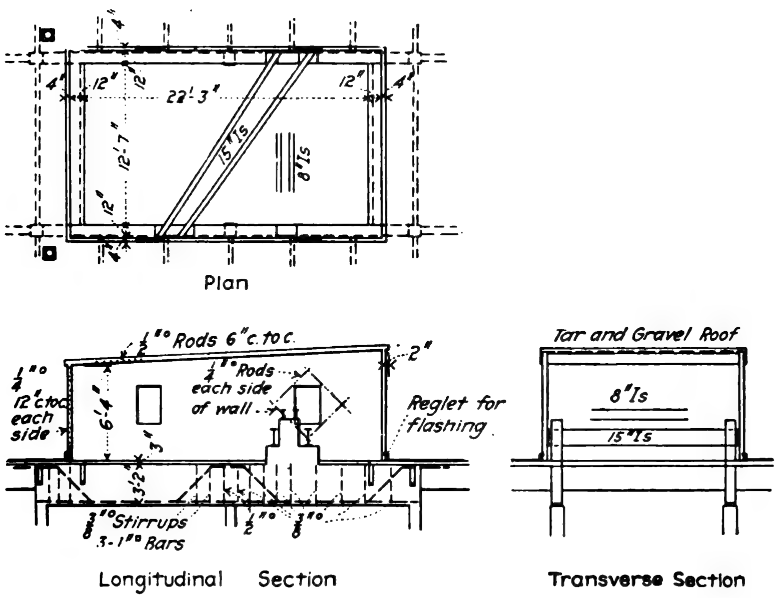


FIG. 275.—Pent-house details.

through special openings in the pan. This method is generally employed when it is advantageous to gain headroom in the basement underneath the pit pans. All footings for columns and foundations adjoining elevator shafts should be kept below the floor of the elevator-shaft pit, and the pit itself should be finished to plumb-line dimensions.

In localities where the water level in the soil is very high, or where there are underground springs, the pits should be thoroughly waterproofed. This may be done in the same manner as for basement floors. (See Art. 29.) If the water pressure is very great, waterproof pans may be employed, as shown in Fig. 274.

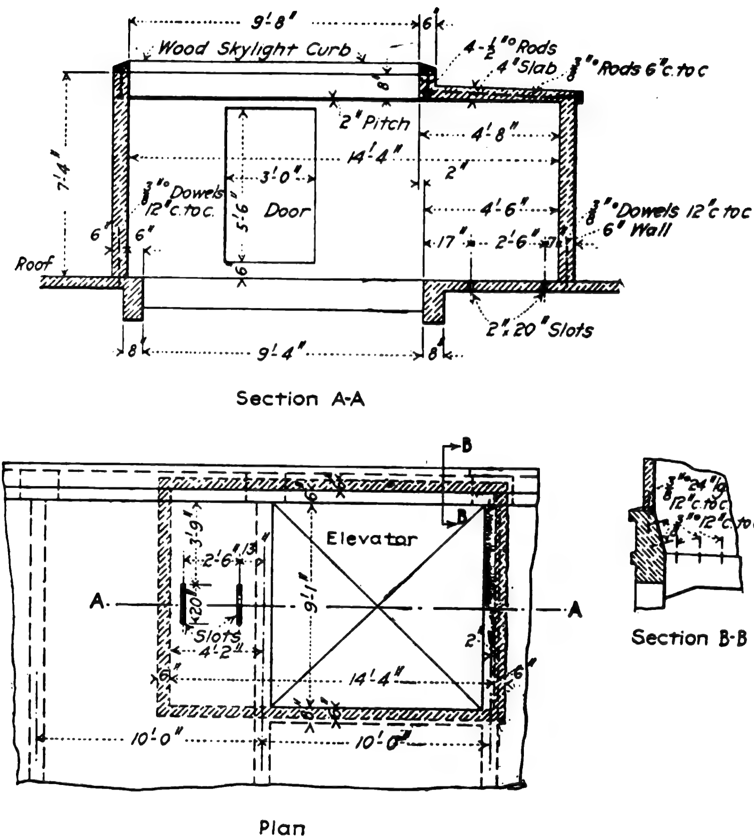


Fig. 276.—Pent-house details.

The sides of the pits are usually made of concrete 6 to 8 in. thick and the bottom of the pit is made a slab of the same thickness as the basement floor. (See Fig. 232.) The sides of the pit are supported at the top and bottom, and may be figured as a simple slab acted upon by the earth pressure and by a pressure due to the live load on the basement floor. The steel used in

the side walls should be run into the basement floor in order to take the reaction at the upper end of the slab.

66. Pent Houses.—The heights of pent houses over elevator shafts vary according to the number of sets of sheave beams and the size of the sheaves. If the machine is located in the basement or on any intermediate floor, the height of the pent house will vary according to the location of the counterweights with respect to the machines. If the machine is placed over the shaft, the size and height of the pent house may be affected by the dimensions of the machine itself. Figs. 275 and 276 give details of actual pent-house constructions.

In all machine rooms, doors should be provided of sufficient size to permit any part of the machinery to be removed in case of repairs. The construction should also be such as to admit of easy access for the purpose of oiling sheave bearings, etc.

Local regulations of some localities require that a grating be installed in all elevator shafts, just below the overhead machinery. This grating should be constructed and properly supported to sustain the weight of a number of men.

CHAPTER XII

PROVISION FOR CONTRACTION AND EXPANSION

67. Methods Employed.—There is no well-established practice of providing for contraction and expansion in reinforced concrete buildings as a whole. Many engineers construct curtain walls into keyways leaving only the wall beams and wall girders subject to contraction, and it has been found that these can usually be reinforced sufficiently to prevent cracking. Temperature reinforcement in wall beams should be placed horizontally near the outer surface and should be well distributed throughout the beam depth. This longitudinal reinforcement should be well lapped at the corners of the building. Mr. Chester J. Hogue, Constructing Engineer for the Concrete Engineering Co., Boston, Mass., states that buildings built by his company at Haverhill, Mass., in the fall of 1911, are 300 ft. in length, have no expansion joints, and have not yet cracked. (The Lang Building previously described is one of the buildings referred to.)

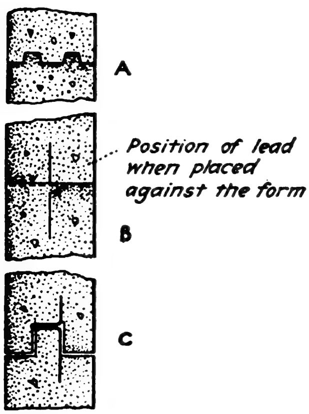


FIG. 277.

Expansion joints to be a success should completely separate the building from bottom to top. They should preferably be made by means of a double column supporting a double girder, and the joints should be waterproofed. Some engineers specify that a complete separation shall be made every 100 ft. in length of the building.

Rather than provide sufficient steel to take contraction in long walls, expansion joints are often resorted to. Fig. 277 shows different methods of making such joints. In type *A* grooves are simply cast in the end of one section and coated with cold-water paint or pitch. In building the next section a tongue is formed fitting the groove. This type of joint is

generally reinforced with burlap when placed in front of an earth fill. (See Fig. 235.) In type *B*, Fig. 277, sheet lead is used. The projecting portion of the sheet is bent up against the form while concrete is being placed in the first section. After re-

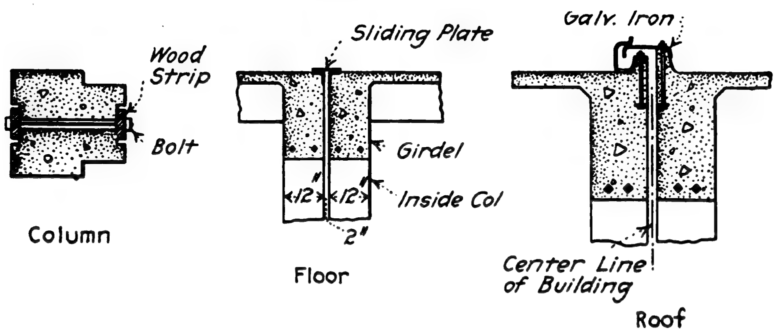


FIG. 278.

moving the forms it is bent down parallel with the wall surface so as to extend into the abutting concrete. A U-shaped bend is formed in the lead sheet at the concrete joint. In type *C* the tongue and groove is formed as in *A*, and in addition a bent lead or copper strip is inserted.

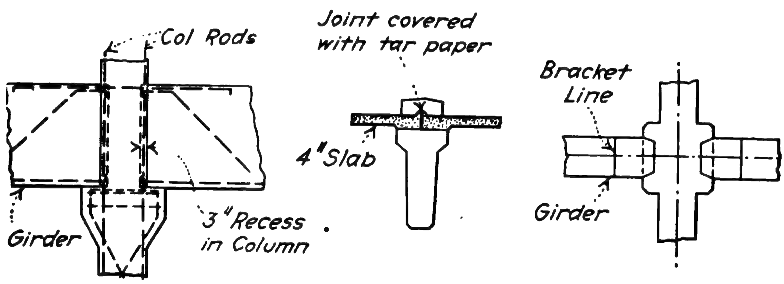


FIG. 279.

Fig. 278 shows the manner of providing expansion joints in the United Shoe Machinery Co.'s factory at Beverly, Mass.

Fig. 279 shows expansion joints in a sawmill at Waycross, Ga., built by the Hebard Cypress Co. An expansion joint occurs directly on every column line. This was done by pouring columns and all transverse beams on column line monolithic, and by making all intermediate beams and girders distinct members. Recesses were made to receive these. The slab was also an

after consideration and was poured independently of beams and girders. The walls were poured after the columns had been stripped. The monolithic feature of concrete construction was entirely eliminated. This was done not only for contraction and expansion, but to permit the pouring of the concrete at convenient times. The roof slabs were cut on all column and ridge lines for movement, and afterward capped. No other covering than concrete was used for the roof and this was waterproofed by introducing a foreign ingredient into the concrete mixture.

The following taken from the *Engineering Record*, issue of March 16, 1912, should be of interest:

"Severe cracks developed in the exterior piers of a large reinforced-concrete building during construction, which, according to the consulting engineers on the work, were due to a rather unusual cause. The building was 600 ft. long, 60 ft. wide, and 4 stories high, without an expansion joint, and the upper stories were built mainly during the summer months. An investigation conducted subsequent to the appearance of the cracks showed that the piers at the center of the building were perfectly sound, while with increasing distances from the center the piers were more seriously affected, those at the ends of the building showing the greatest displacement. As this condition suggested that temperature expansion was responsible for the trouble, measurements were taken by plumbing down from the third floor level to the basement floor. These measurements showed that the third floor expanded, under the direct rays of the sun during a hot summer's day, a sufficient amount to swing the columns out from a perpendicular to an inclined position. The amount of this displacement, it is stated, was in some cases as high as 1½ in. at each end of the building."

CHAPTER XIII

SHEAR AND MOMENT CONSIDERATIONS IN CONTINUOUS BEAMS

It is the intention in this chapter to give the facts which have led to the selection of the general bending moment coefficients as recommended by the Joint Committee, and at the same time to give sufficient information and data whereby the student may be able to design continuous beams for special cases. In ordinary building construction, the span lengths of continuous beams do not vary greatly and the recommended bending moments are entirely satisfactory. But cases sometimes arise—such as the occurrence of unequal spans, spans of unusual length, and instances of heavy concentrated loading—which require special calculations to be made, and the writer feels that the student should have sufficient grasp of the method of figuring continuous beams to be able to cope successfully with such cases.

The continuous beam formulas treated in the next article apply to beams with a uniform moment of inertia throughout. Although this is not the case in continuous beams of reinforced concrete, the formulas given apply directly to either rectangular or T-beams having approximately the same amount of steel over supports as at the center of span.

68. Beams with Uniform Moment of Inertia.—As explained in Art. 54 of Volume I, the bending moments and shears of continuous beams cannot be derived from the principles of statics, but they may be determined from the relation which exists between bending moment and deflection at any given section of a loaded beam. The position taken by the neutral surface under loading is called the elastic curve and the deflection at any given point is the ordinate between this curve and the neutral surface of the unbent beam.

An attempt will be made to give a general outline of the steps taken in the derivation of the *theorem of three moments*—the theorem by which continuous beams are figured. The student who understands calculus will appreciate the method in its entirety, but it is thought that the student without this mathe-

matical preparation will obtain a sufficient idea of the elements involved to be able to appreciate the application of the above-named theorem. For the student who has already studied the methods of figuring continuous beams, the following may prove a helpful review.

Consider first a homogeneous beam. In Fig. 280 let Δl represent the original distance between the planes $ABCD$ and $EFGH$, or the distance MS in the figure. Consider a filament of cross-sectional area $\Delta\ A$ (Δ in every case will be taken to mean an extremely small part of the total) connecting these planes at a distance r above the neutral surface. In the part of the beam

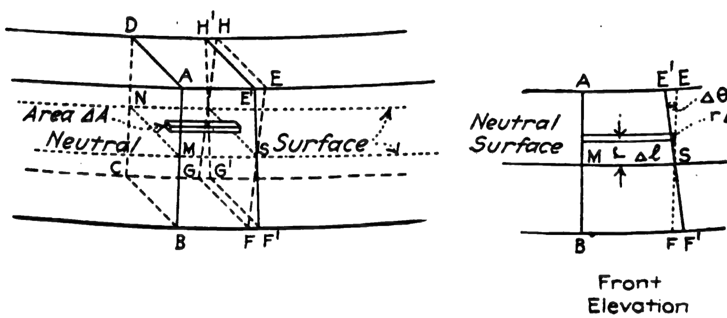


FIG. 280.

concave upward (the case shown) the filament is shortened an amount $r\Delta\theta$, expressing the angle of shortening ($\Delta\theta$) in circular measure. Likewise for a filament similarly placed below the neutral surface, there is a lengthening of the fibers by this same amount. The unit deformation is the ratio of the total deformation to the original length, and thus equals $\frac{r\Delta\theta}{\Delta l}$. The unit stress in a filament is the product of the unit deformation by the modulus of elasticity, or

$$f = Er\frac{\Delta\theta}{\Delta l}$$

The total stress on a filament of area ΔA is the product of the unit stress by the area; that is,

$$\text{total stress on } \Delta A = Er\frac{\Delta\theta}{\Delta l}\ (\Delta A)$$

The moment of this stress with respect to the neutral axis MN

is the product of the total stress on $\triangle A$ by the moment arm r, or

$$\text{moment of stress about axis} = Er^2\,(\triangle A)\,\frac{\Delta\theta}{\Delta l}$$

The total moment of all the filaments which make up the beam is the summation of all such bending moments over the section $ABCD$. By the definition of the moment of inertia of a plane surface, the summation of $r^2(\triangle A)$ is equal to I, or

$$M = EI\frac{\Delta\theta}{\Delta l} \tag{1}$$

In a reinforced-concrete beam the I for steel would be taken n times that for an equal area of concrete situated at the same distance from the neutral axis. It should be noted that $\triangle\theta$ in the above equation is the change in slope of the tangent to the neutral surface in the length $\triangle l$.

Fig. 281 shows the neutral surface of a continuous beam with the vertical deflection exaggerated. A point will be chosen with coördinates x and y, and with x starting from the support A at

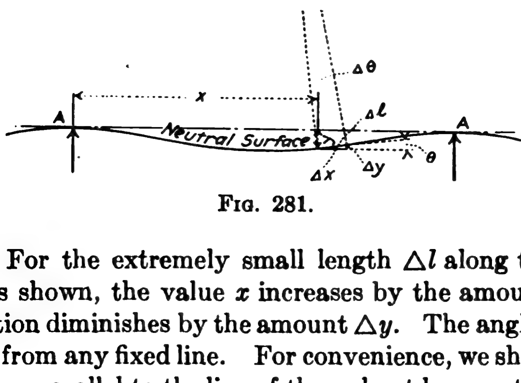


FIG. 281.

the left. For the extremely small length $\triangle l$ along the neutral surface, as shown, the value x increases by the amount $\triangle x$ and the deflection diminishes by the amount $\triangle y$. The angle θ may be measured from any fixed line. For convenience, we shall measure θ from a line parallel to the line of the unbent beam—that is, AA. The angle θ is then the angle which the tangent to the curved beam makes with the original direction of the beam. The angle $\triangle\theta$ is the change made in this angle for an extremely short distance $\triangle l$ along the neutral surface, and $\frac{\Delta\theta}{\Delta l}$ is the rate of change of the angle θ at the given section.

Referring to Fig. 281 it is seen that where $\triangle x$ and $\triangle y$ are considered extremely small

$$\tan\theta = \frac{\Delta y}{\Delta x} \tag{2}$$

Also

$$\Delta x = \Delta l \cos \theta \tag{3}$$

and

$$\cos \theta = \frac{1}{\sec \theta} = \frac{1}{\sqrt{1 + \left(\frac{\Delta y}{\Delta x}\right)^2}} \tag{4}$$

It is now possible by calculus, making use of the four equations given above, to obtain an expression for the elastic curve by which the deflection y at any given section may be determined, provided the bending moment, modulus of elasticity, and moment of inertia at this section are known. In a continuous beam, however, moment is not known nor can it be found directly. Fortunately the equation for the elastic curve above mentioned gives us the necessary help to find the moments at the supports, since for one thing we know that deflection is zero at these points. In fact, we have been discussing deflection only in order to express the relation of the moments at successive supports of a continuous beam in terms of the length of the intervening spans and the loads which they carry.

To indicate in a general way how this may be done, consider any two consecutive spans. The equation of the elastic curve as expressed for any given section in the left span will contain the value of the moment at the left support modified by the loads between this support and the given section. Likewise for any given section in the span to the right, the equation of the elastic curve includes the value of the moment at the middle support modified by the loads between this support and the section in question. Knowing among other things that the deflection is zero at the supports and that the rate of change in slope of the tangent to the neutral surface at the center support with respect to both spans is identical, it is possible to derive an expression containing the moments at the three supports, together with the lengths of the two intermediate spans and their loading. This expression is known as the *theorem of three moments*. In beams of uniform moment of inertia, as assumed, the terms E and I are the same throughout both spans and they cancel. In beams with variable moment of inertia, however, this is not the case and a very complex expression results.

THEOREM OF THREE MOMENTS

By means of the theorem above mentioned, the moments at the supports of a continuous beam may be deduced. The theorem, which assumes level supports and uniform section, is in its general form as follows (Fig. 282):

$$M_2l_2+2M_3(l_2+l_3)+M_4l_3 = -\Sigma P_2l_2^2(k-k^3)-\Sigma P_3l_3^2(2k-3k^2+k^3) \quad (5)$$

As above expressed, the theorem refers to the spans M_2-M_3 and M_3-M_4. By increasing all subscripts by unity it will refer to spans M_3-M_4 and M_4-M_5, and so on. Each equation includes one new moment and it is in this manner, not consider-

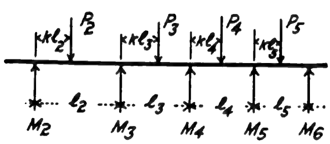


FIG. 282.

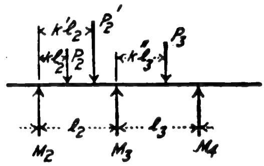


FIG. 283.

ing beams with fixed ends for the present, that there are obtained as many equations as there are unknown moments—the end moments being known. Where there is no overhang, end moments are zero. The theorem in its general form, as given above, applies directly to concentrated loads and Σ preceding the last two terms means the summation of these terms for the various loads which act in the spans considered. The value of k changes for each load.

Suppose, for example, that it is desired to write an equation for span M_2-M_3 and M_3-M_4, Fig. 283, with loads on the two spans as shown. Then, the theorem may be written for this case as follows:

$$M_2l_2+2M_3(l_2+l_3)+M_4l_3 = -P_2l_2^2(k-k^3)- P'_2l_2^2(k'-k'^3)-P_3l_3^2(2k''-3k''^2+k''^3).$$

For two spans uniformly loaded, the theorem has the following form:

$$M_2l_2+2M_3(l_2+l_3)+M_4l_3 = -\tfrac{1}{4}w_2l_2^3-\tfrac{1}{4}w_3l_3^3 \quad (6)$$

where $\begin{pmatrix} w_2 \\ w_3 \end{pmatrix}$ = the uniform load per foot in span $\begin{pmatrix} l_2 \\ l_3 \end{pmatrix}$. If there is no load in span l_2, then w_2 becomes zero; likewise with w_3.

When the spans of a continuous girder are all equal, the expression for concentrated loads becomes, by dividing through by the length of span,

$$M_2+4M_3+M_4=-\Sigma P_2l(k-k^3)-\Sigma P_3l(2k-3k^2+k^3) \qquad (7)$$

and, for two spans uniformly loaded with the same load w per linear foot,

$$M_2+4M_3+M_4=-\tfrac{1}{2}wl^2 \qquad (8)$$

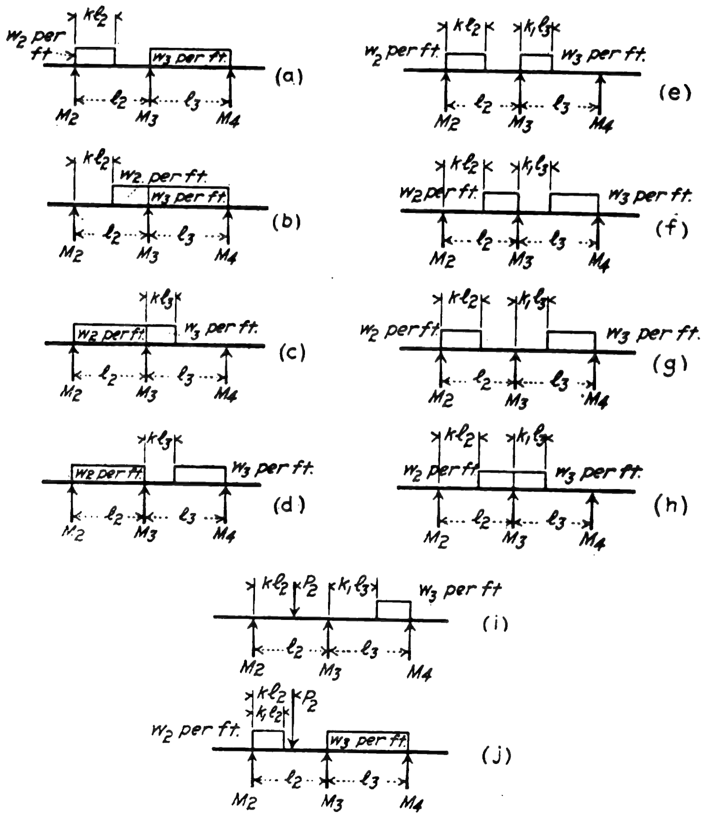


FIG. 284.

When either span (or both) is only partially covered with a uniform load, the expression for the theorem to the right of the equality sign takes other forms. Fig. 284 shows the various cases which may arise, and below are given the corresponding expressions. If no load occurs in a span considered as fully loaded in

the figure, then the term in the theorem to the right of the equality sign representing this span becomes zero. The proper value of k is to be substituted in each case.

Let $M_2l_2+2M_3(l_2+l_3)+M_4l_3$ be denoted by r.

Then (a) $r=-w_2l_2^3\left(\frac{k^2}{2}-\frac{k^4}{4}\right)-\frac{1}{4}w_3l_3^3.$

(b) $r=-w_2l_2^3\left(\frac{1}{4}-\frac{k^2}{2}+\frac{k^4}{4}\right)-\frac{1}{4}w_3l_3^3.$

(c) $r=-\frac{1}{4}w_2l_2^3-w_3l_3^3\left(k^2-k^3+\frac{k^4}{4}\right).$

(d) $r=-\frac{1}{4}w_2l_2^3-w_3l_3^3\left(\frac{1}{4}-k^2+k^3-\frac{k^4}{4}\right).$

(e) $r=-w_2l_2^3\left(\frac{k^2}{2}-\frac{k^4}{4}\right)-w_3l_3^3\left(k_1^2-k_1^3+\frac{k_1^4}{4}\right).$

(f) $r=-w_2l_2^3\left(\frac{1}{4}-\frac{k^2}{2}+\frac{k^4}{4}\right)-w_3l_3^3\left(\frac{1}{4}-k_1^2+k_1^3-\frac{k_1^4}{4}\right).$

(g) $r=-w_2l_2^3\left(\frac{k^2}{2}-\frac{k^4}{4}\right)-w_3l_3^3\left(\frac{1}{4}-k_1^2+k_1^3-\frac{k_1^4}{4}\right).$

(h) $r=-w_2l_2^3\left(\frac{1}{4}-\frac{k^2}{2}+\frac{k^4}{4}\right)-w_3l_3^3\left(k_1^2-k_1^3+\frac{k_1^4}{4}\right).$

(i) $r=-\Sigma P_2l_2^2\left(k-k^3\right)-w_3l_3^3\left(\frac{1}{4}-k_1^2+k_1^3-\frac{k_1^4}{4}\right).$

(j) $r=-\Sigma P_2l_2^2\left(k-k^3\right)-w_2l_2^3\left(\frac{k_1^2}{2}-\frac{k_1^4}{4}\right)-\frac{1}{4}w_3l_3^3.$

The student should notice that spans similarly located and similarly loaded have the same representative term. Cases (i) and (j) are given to show how concentrated loads may be considered with uniform loads and the student should be able to supply the terms for the other cases of such a combination.

When a continuous beam has fixed ends the theorem of three moments may also be used to determine the bending moments at the supports. In this type of beam there are two more unknown moments then where the ends are simply supported. An additional span should be considered to exist at each end of such beams. Then two more equations may be written than if these spans were not added. After the equations are written, the lengths of the two added spans are made zero and the neutral surface becomes horizontal at the ends of the beam. This is the condition of the neutral surface at fixed ends as pointed out in Art. 53 of Volume I.

In order to show how a continuous beam may be figured by

the theorem of three moments, consider a beam of three spans with ends simply supported, as shown in Fig. 285(a). The beam is loaded with a uniform load over the entire structure. The equations which may be written for each support are as follows (see formula 8):

$$M_1+4M_2+M_3=-\tfrac{1}{2}wl^2$$
$$M_2+4M_3+M_4=-\tfrac{1}{2}wl^2$$

Since the ends of the beam merely rest upon the supports, $M_1=M_4=0$. Hence the two equations furnish the means of finding

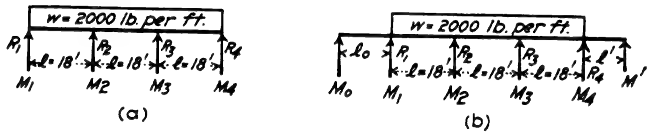


FIG. 285.

the two unknown moments M_2 and M_3. The solution may be abridged by the fact that $M_2=M_3$, which is evident from the symmetry of the beam and of the loading.

$$M_3=-\tfrac{1}{2}wl^2-4M_2$$
$$M_2+4(-\tfrac{1}{2}wl^2-4M_2)=-\tfrac{1}{2}wl^2$$
$$M_2=M_3=-\tfrac{1}{10}wl^2=-64{,}800 \text{ ft.-lb.}$$

Taking the center of moments at R_2,

$$18\,R_1-(2000)\,(18)\,(9)=-64{,}800$$
$$R_1=R_4=14{,}400 \text{ lb.}$$

With the center of moments at R_3,

$$14{,}400(36)+R_2(18)-(2000)(36)(18)=-64{,}800$$
$$R_2=R_3=39{,}600 \text{ lb.}$$

Now that the reactions are all known, the shears and moments at any section may be determined in a manner similar to the ordinary beam.

Consider the ends of the beam to be fixed (Fig. 285-b). The equations become by formulas (6) and (8)

$$M_0l_0+2M_1(l_0+l_1)+M_2l=-\tfrac{1}{4}wl^3$$
$$M_1+4M_2+M_3=-\tfrac{1}{2}wl^2$$
$$M_2+4M_3+M_4=-\tfrac{1}{2}wl^2$$
$$M_3l+2M_4(l+l')+M'l'=-\tfrac{1}{4}wl^3$$

Now, M_0 and M' are zero since the ends are supposed to merely rest upon the supports. Also, by making $l_0=0$ and $l'=0$, the equations become

$$2M_1+M_2=-\tfrac{1}{4}wl^2$$
$$M_1+4M_2+M_3=-\tfrac{1}{2}wl^2$$
$$M_2+4M_3+M_4=-\tfrac{1}{2}wl^2$$
$$M_3+2M_4=-\tfrac{1}{4}wl^2$$

Solving these equations:

$$M_1=M_2=M_3=M_4=-\tfrac{1}{12}wl^2=-54{,}000 \text{ ft.-lb.}$$
$$R_1=R_4=18{,}000 \text{ lb.}$$
$$R_2=R_3=36{,}000 \text{ lb.}$$

For simple beams, the determination of shear (V) and moment (M) for any special section is easily accomplished, as the left reaction can be readily found for any given loads. For continuous beams, however, it is not always such a simple matter, especially when a beam has fixed ends. Let Fig. 286 represent one span of a continuous beam. Let V be the shear for any section at the distance x from the left support, and V' the shear at a section infinitely near to the left support. Also let ΣP_1 denote the sum of all the concentrated loads on the distance x, and wx the uniform load. Then

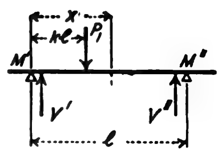


FIG. 286.

$$V=V'-wx-\Sigma P_1.$$

Hence the shear V can be determined for any section in the span as soon as V' is known.

Since the shear V' in Fig. 286 is equal to the resultant of all the vertical forces on the left of a section taken at the right of the left support, let r be the distance of the line of action of that resultant to the left of that support. Then the moment at any section is

$$M=V'(r+x)-wx.\tfrac{1}{2}x-\Sigma P_1(x-kl).$$

But the quantity $V'r$ is the resultant moment of all the forces to the left of the left support, with respect to that support, and equals M'. Hence

$$M=M'+V'x-\tfrac{1}{2}wx^2-\Sigma P_1(x-kl) \qquad (9)$$

from which M can be found knowing M' and V'.

The shear V' may be easily determined if M' and M'' are known. Thus in (9) make $x=l$, then M becomes M'', and

$$V'=\frac{M''-M'}{l}+\tfrac{1}{2}wl+\Sigma P_1(1-k) \tag{10}$$

Uniform Load over All Spans

In Fig. 287 are given the moments in continuous beams for a uniform load over all spans. Supported ends and equal spans are assumed. The shears on each side of the supports are given

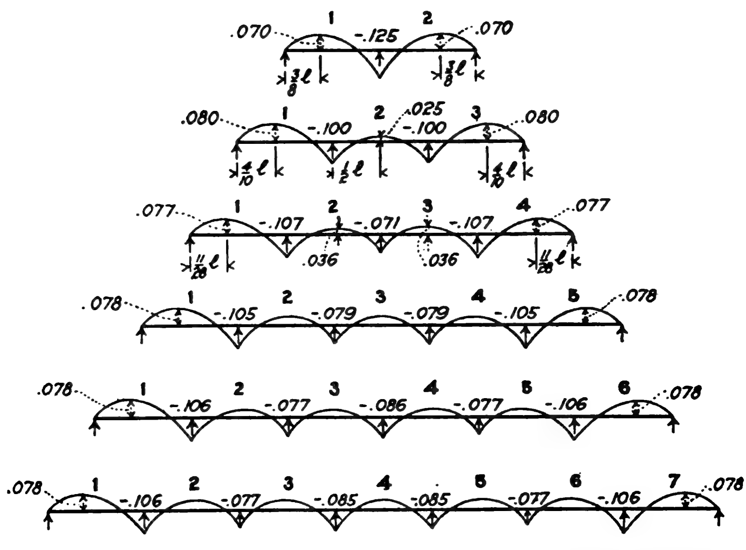


FIG. 287.—Moments in continuous beams; supported ends; uniform load on all spans; spans all equal. Coefficients of (wl^2).

in Fig. 288. The reaction at any given support is the sum of the two shears at that support.

The maximum moments at (or near) the center of many of the interior spans are not given, as they are small and do not vary greatly from those given for the continuous beam of four spans. Maximum positive moment occurs for zero shear as in simple beams.

In continuous beams with fixed ends, and with uniform load

and equal spans (the same as above), the maximum positive moment occurs at the center of each span and its value in every case is $\frac{wl^2}{24}$. The negative moment over each support equals $\frac{wl^2}{12}$. Shears close to the supports are all equal with a value of $\frac{1}{2}$ *wl.*

Fixed and Moving Concentrated Loads

Data sheets 1 to 7 inclusive give the preliminary work which is necessary for the plotting of influence lines[1] for continuous beams —both fixed and supported ends. The tables, which are made

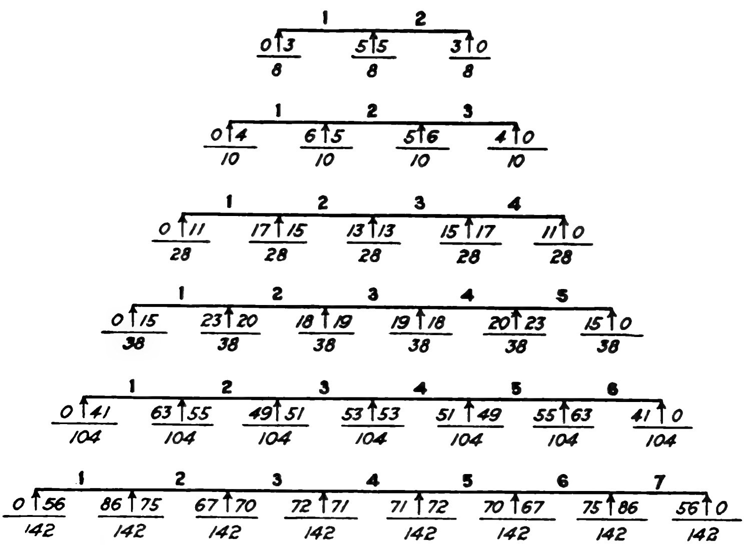


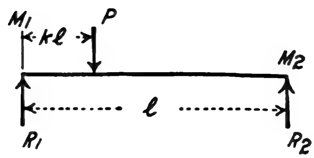
Fig. 288.—Shears in continuous beams; supported ends; uniform loads on all spans; spans all equal. Coefficients of (wl).

out for 10-ft. spans, are needed for the convenient plotting of results. Figs. 289 to 293 inclusive are the corresponding influence lines. Continuous beams of one, two, and three equal spans and any span length may be figured for concentrated loads by means of these lines as hereafter explained.

[1] Influence lines are treated in "The Elements of Structures."

DATA SHEET NO. 1

One Span—Fixed Ends
Concentrated load as shown.



$$M_1 = -Plk(1-2k+k^2)$$
$$M_2 = -Plk^2(1-k)$$
$$R_1 = P(1-3k^2+2k^3)$$
$$R_2 = Pk^2(3-2k)$$

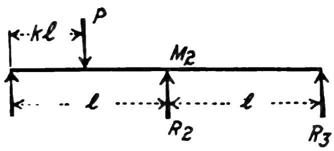
Spans, 10 ft. Load, unity

	Values of k									
	0.1	0.2	0.3	0.4	0.45	0.5	0.6	0.7	0.8	0.9
M_1	−0.810	−1.280	−1.470	−1.440	−1.363	−1.250	−0.960	−0.630	−0.320	−0.090
R_1	0.972	0.896	0.784	0.648	0.575	0.500	0.352	0.216	0.104	0.028

DATA SHEET NO. 2

Two Spans—Supported Ends

Concentrated load on left-end span as shown. Let B denote the quantity $(k-k^3)$



$$M_2 = -\frac{1}{4}Pl(B)$$

$$R_1 = P(1-k) - \frac{P}{4}(B)$$

$$R_2 = Pk + \frac{P}{2}(B)$$

$$R_3 = -\frac{P}{4}(B)$$

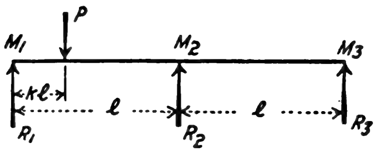
Spans, 10 ft. Load, unity

	Values of k										
	0.1	0.2	0.3	0.35	0.4	0.45	0.5	0.6	0.7	0.8	0.9
M_2	−0.247	−0.480	−0.682	....	−0.840	....	−0.937	−0.960	−0.892	−0.720	−0.427
R_1	0.875	0.752	0.632	0.573	0.516	0.460	0.406	0.304	0.211	0.128	0.057
R_2	0.149	0.296	0.437	....	0.568	....	0.687	0.792	0.879	0.944	0.985
R_3	−0.025	−0.048	−0.068	....	−0.084	....	−0.094	−0.096	−0.089	−0.072	−0.043

DATA SHEET NO. 3

Two Spans—Fixed Ends

Concentrated load on left-end span as shown. Let A denote the quantity $(2k-3k^2+k^3)$ and let B denote the quantity $(k-k^3)$



$$M_1=-\frac{Pl}{12}(7A-2B) \qquad R_1=P(1-k)+\frac{P}{4}(3A-2B)$$

$$M_2=\frac{Pl}{6}(A-2B) \qquad R_2=Pk-P(A-B)$$

$$M_3=-\frac{Pl}{12}(A-2B) \qquad R_3=\frac{P}{4}(A-2B)$$

Spans, 10 ft. Load, unity

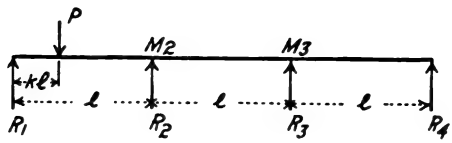
Values of k

	0.1	0.2	0.3	0.4	0.45	0.5	0.55	0.6	0.7	0.8	0.9
M_1	−0.832	−1.360	−1.627	−1.680	−1.639	−1.562	−1.454	−1.320	−0.997	−0.640	−0.292
M_2	−0.045	−0.160	−0.315	−0.480	....	−0.625	....	−0.720	−0.735	−0.640	−0.405
M_3	0.023	0.080	0.157	0.240	....	0.312	....	0.360	0.367	0.320	0.202
R_1	0.979	0.920	0.831	0.720	0.658	0.594	0.527	0.460	0.326	0.200	0.069
R_2	0.028	0.104	0.216	0.352	....	0.500	....	0.648	0.784	0.896	0.972
R_3	−0.007	−0.024	−0.047	−0.072	....	−0.094	....	−0.108	−0.110	−0.094	−0.061

DATA SHEET NO. 4

Three Spans—Supported Ends

Concentrated load on left-end span as shown. *B* as before.



$$M_2 = -\frac{4}{15}Pl(B) \qquad R_2 = Pk + \frac{3}{5}P(B)$$

$$M_3 = \frac{Pl}{15}(B) \qquad R_3 = -\frac{2}{5}P(B)$$

$$R_1 = P(1-k) - \frac{4}{15}P(B) \qquad R_4 = \frac{P}{15}(B)$$

Spans, 10 ft. Load, unity

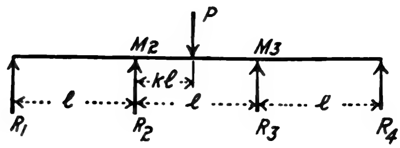
Values of k

	0.1	0.2	0.3	0.35	0.4	0.45	0.5	0.6	0.7	0.8	0.9
M_2	−0.264	−0.512	−0.728	....	−0.896	....	−1.000	−1.024	−0.952	−0.768	−0.456
	0.066	0.128	0.182	....	0.224	....	0.250	0.256	0.238	0.192	0.114
	0.874	0.749	0.627	0.568	0.510	0.454	0.400	0.298	0.205	0.123	0.054
	0.159	0.315	0.464	....	0.602	....	0.725	0.830	0.914	0.973	1.003
	.040	−0.077	−0.109	....	−0.134	....	−0.150	−0.154	−0.143	−0.115	−0.068
	07	0.013	0.018	....	0.022	....	0.025	0.026	0.024	0.019	0.011

DATA SHEET NO. 5

Three Spans—Supported Ends (Continued)

Concentrated load on center span as shown. *A* and *B* as before.



$$M_2 = -\frac{Pl}{15}(4A - B) \qquad R_2 = P(1-k) + \frac{P}{5}(3A - 2B)$$

$$M_3 = \frac{Pl}{15}(A - 4B) \qquad R_3 = Pk - \frac{P}{5}(2A - 3B)$$

$$R_1 = -\frac{P}{15}(4A - B) \qquad R_4 = \frac{P}{15}(A - 4B)$$

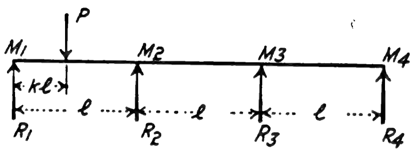
Spans, 10 ft. Load, unity

	Values of k									
	0.1	0.2	0.3	0.4	0.45	0.5	0.6	0.7	0.8	0.9
M_2	−0.390	−0.640	−0.770	−0.800	−0.784	−0.750	−0.640	−0.490	−0.320	−0.150
M_3	−0.150	−0.320	−0.490	−0.640	−0.701	−0.750	−0.800	−0.770	−0.640	−0.390
R_1	−0.039	−0.064	−0.077	−0.080	−0.078	−0.075	−0.064	−0.049	−0.032	−0.015
R_2	0.963	0.896	0.805	0.696	0.637	0.575	0.448	0.321	0.200	0.091
R_3	0.091	0.200	0.321	0.448	....	0.575	0.696	0.805	0.896	0.963
R_4	−0.015	−0.032	−0.049	−0.064	....	−0.075	−0.080	−0.077	−0.064	−0.039

DATA SHEET NO. 6

Three Spans—Fixed Ends

Concentrated load on left-end span as shown. A and B as before.



$$M_1 = -\frac{Pl}{45}(26A - 7B) \qquad R_1 = P(1-k) + \frac{P}{15}(11A - 7B)$$

$$M_2 = \frac{7}{45}Pl(A - 2B) \qquad R_2 = Pk - \frac{P}{15}(14A - 13B)$$

$$M_3 = -\frac{2}{45}Pl(A - 2B) \qquad R_3 = \frac{4}{15}P(A - 2B)$$

$$M_4 = \frac{Pl}{45}(A - 2B) \qquad R_4 = -\frac{P}{15}(A - 2B)$$

Spans, 10 ft. Load, unity

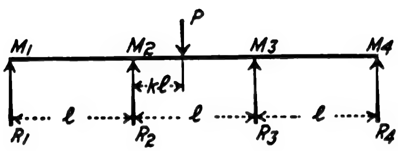
Values of k

	0.1	0.2	0.3	0.4	0.45	0.5	0.55	0.6	0.7	0.8	0.9
M_1	−0.834	−1.365	−1.638	−1.674	−1.658	−1.583	−1.477	−1.344	−1.022	−0.661	−0.306
M_2	−0.042	−0.149	−0.294	−0.448	....	−0.583	....	−0.672	−0.686	−0.597	−0.378
M_3	0.012	0.043	0.084	0.128	....	0.167	....	0.192	0.196	0.171	0.108
M_4	−0.006	−0.021	−0.042	−0.064	....	−0.083	....	−0.096	−0.098	−0.085	−0.054
R_1	0.979	0.922	0.834	0.725	0.664	0.600	0.534	0.467	0.334	0.206	0.093
R_2	0.026	0.098	0.203	0.333	....	0.475	....	0.619	0.755	0.870	0.956
R_3	−0.007	−0.026	−0.050	−0.077	....	−0.100	....	−0.115	−0.118	−0.102	−0.065
R_4	0.002	0.006	0.013	0.019	....	0.025	....	0.029	0.029	0.026	0.016

DATA SHEET NO. 7

Three Spans—Fixed Ends (Continued)

Concentrated load on center span as shown. A and B as before.



$$M_1 = \frac{Pl}{45}(7A - 2B) \qquad R_1 = -\frac{P}{15}(7A - 2B)$$

$$M_2 = -\frac{2}{45}Pl(7A - 2B) \qquad R_2 = P(1-k) + \frac{P}{15}(13A - 8B)$$

$$M_3 = \frac{2}{45}Pl(2A - 7B) \qquad R_3 = Pk - \frac{P}{15}(8A - 13B)$$

$$M_4 = -\frac{Pl}{45}(2A - 7B) \qquad R_4 = \frac{P}{15}(2A - 7B)$$

Spans, 10 ft. Load, nuity

Values of k

	0.1	0.2	0.3	0.4	0.45	0.5	0.6	0.7	0.8	0.9
M_1	0.222	0.363	0.434	0.448	....	0.417	0.352	0.266	0.171	0.078
M_2	−0.444	−0.726	−0.868	−0.896	−0.874	−0.833	−0.704	−0.532	−0.342	−0.156
M_3	−0.156	−0.342	−0.532	−0.704	−0.776	−0.833	−0.896	−0.868	−0.726	−0.444
M_4	0.078	0.171	0.266	0.352	....	0.417	0.448	0.434	0.363	0.222
R_1	−0.067	−0.109	−0.130	−0.134	−0.131	−0.125	−0.102	−0.080	−0.051	−0.023
R_2	0.995	0.947	0.864	0.754	0.691	0.625	0.486	0.346	0.213	0 095
R_3	0.095	0.213	0.346	0.486	....	0.625	0.754	0.864	0.947	0.995
R_4	−0.023	−0.051	−0.080	−0.102	....	−0.125	−0.134	−0.130	−0.109	−0.067

Values of $(k-k^3)$ and $(2k-3k^2+k^3)$

Read down for $B = (k-k^3)$

	0	1	2	3	4	5	6	7	8	9		
0.0	0.0000	0100	0200	0300	0399	0499	0598	0697	0795	0893	0990	0.9
0.1	0.0990	1087	1183	1278	1373	1466	1559	1651	1742	1831	1920	0.8
0.2	0.1920	2007	2094	2178	2262	2344	2424	2503	2580	2656	2730	0.7
0.3	0.2730	2802	2872	2941	3007	3071	3134	3193	3251	3307	3360	0.6
0.4	0.3360	3411	3459	3505	3548	3589	3627	3662	3694	3724	3750	0.5
0.5	0.3750	3773	3794	3811	3825	3836	3844	3848	3849	3846	3840	0.4
0.6	0.3840	3830	3817	3800	3779	3754	3725	3692	3656	3615	3570	0.3
0.7	0.3570	3521	3468	3410	3348	3281	3210	3135	3054	2970	2880	0.2
0.8	0.2880	2786	2686	2582	2473	2359	2239	2115	1985	1850	1710	0.1
0.9	0.1710	1564	1413	1256	1094	0926	0753	0573	0388	0197	0000	0.0
		9	8	7	6	5	4	3	2	1	0	

Read up for $A = (2k-3k^2+k^3)$

The equations given on the Data Sheets were all obtained by means of the Theorem of Three Moments. The table which follows the Data Sheets helps materially in solving these equa-

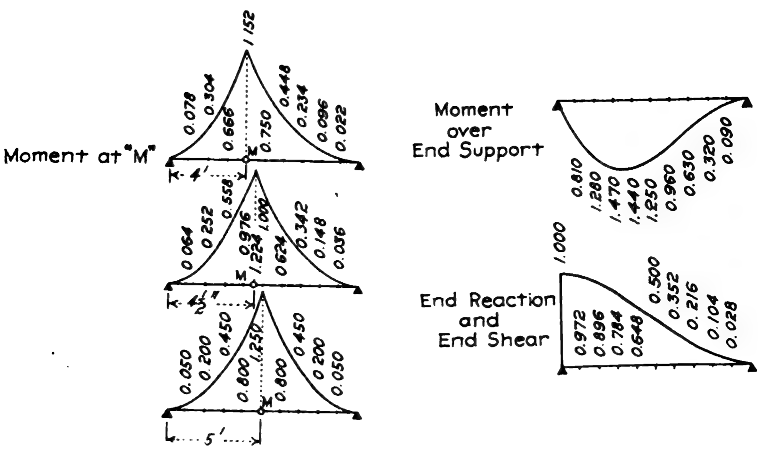


Fig. 289.—Influence lines for one span, fixed ends. (Spans, 10 ft. Load, unity.)

tions for given values of k. To the maximum values of the functions determined for the live concentrated loads must be added the effect of the dead load (uniform load over all spans, Figs. 287 and 288) in order to obtain the total maximum values. No matter where the maximum positive moment occurs in a

given span of a continuous beam it is sufficiently accurate to add the maximum positive dead-load moment in that span.

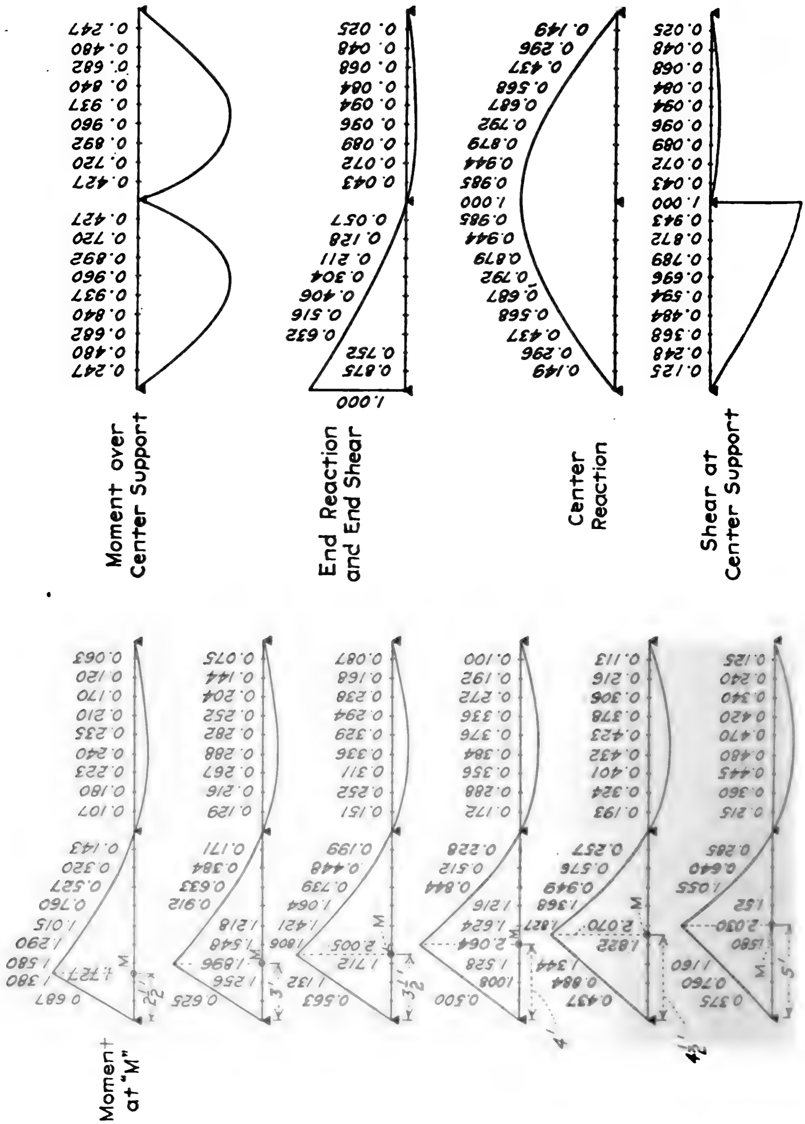


FIG. 290.—Influence lines for two equal spans, supported ends. (Spans, 10 ft. Load, unity.)

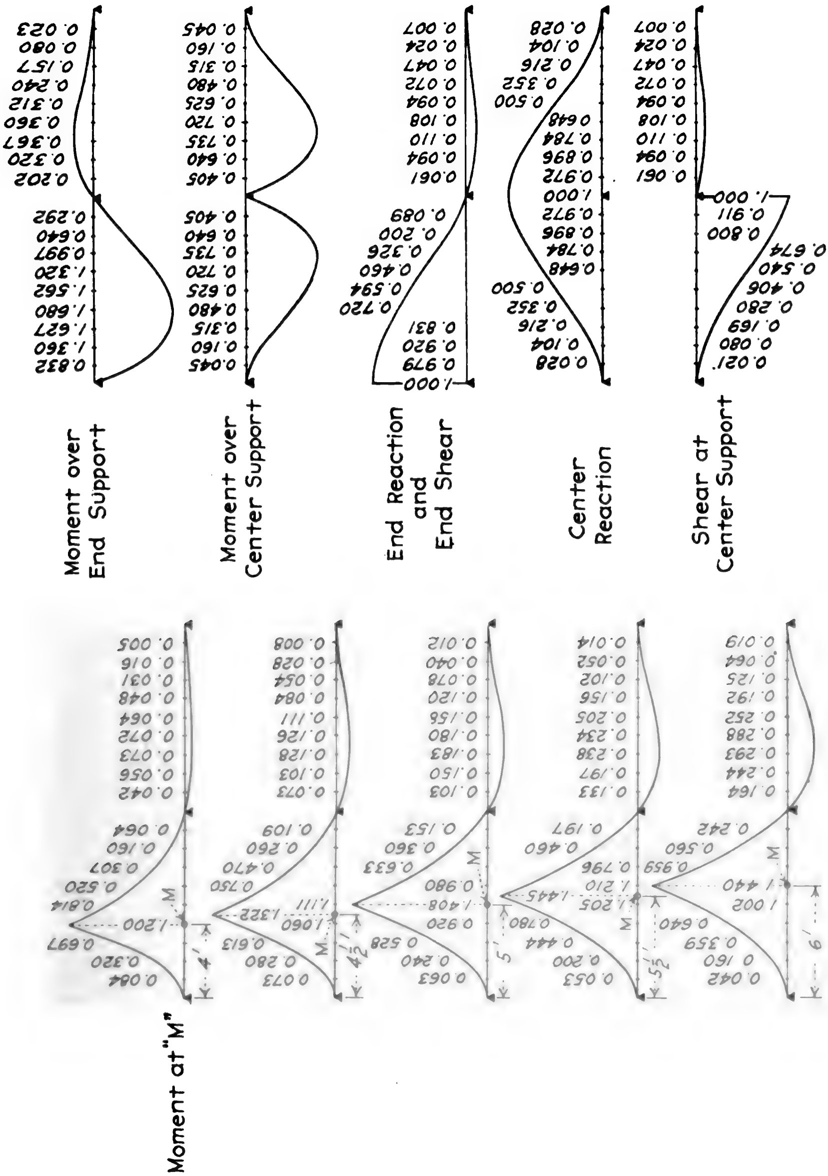


FIG. 291.—Influence lines for two equal spans, fixed ends. (Spans, 10 ft. Load, unity.)

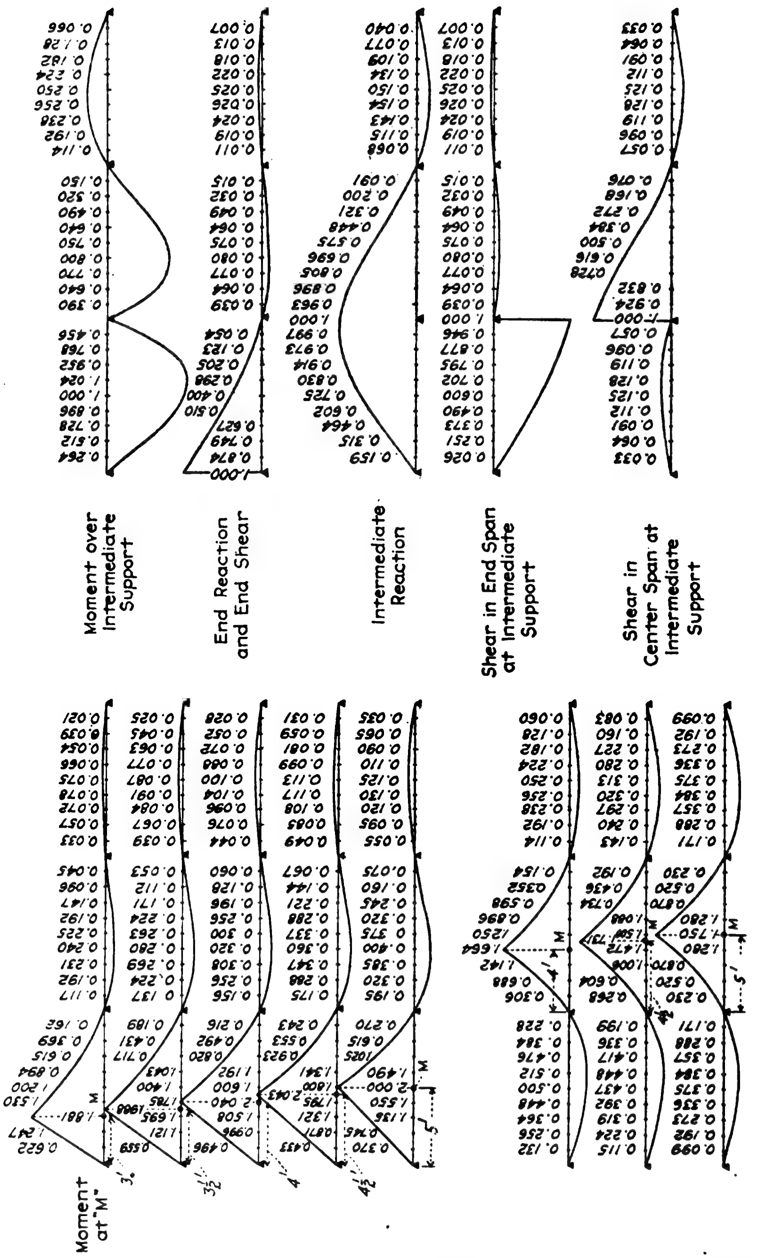


FIG. 292.—Influence lines for three equal spans, supported ends. (Spans, 10 ft. Load, unity.)

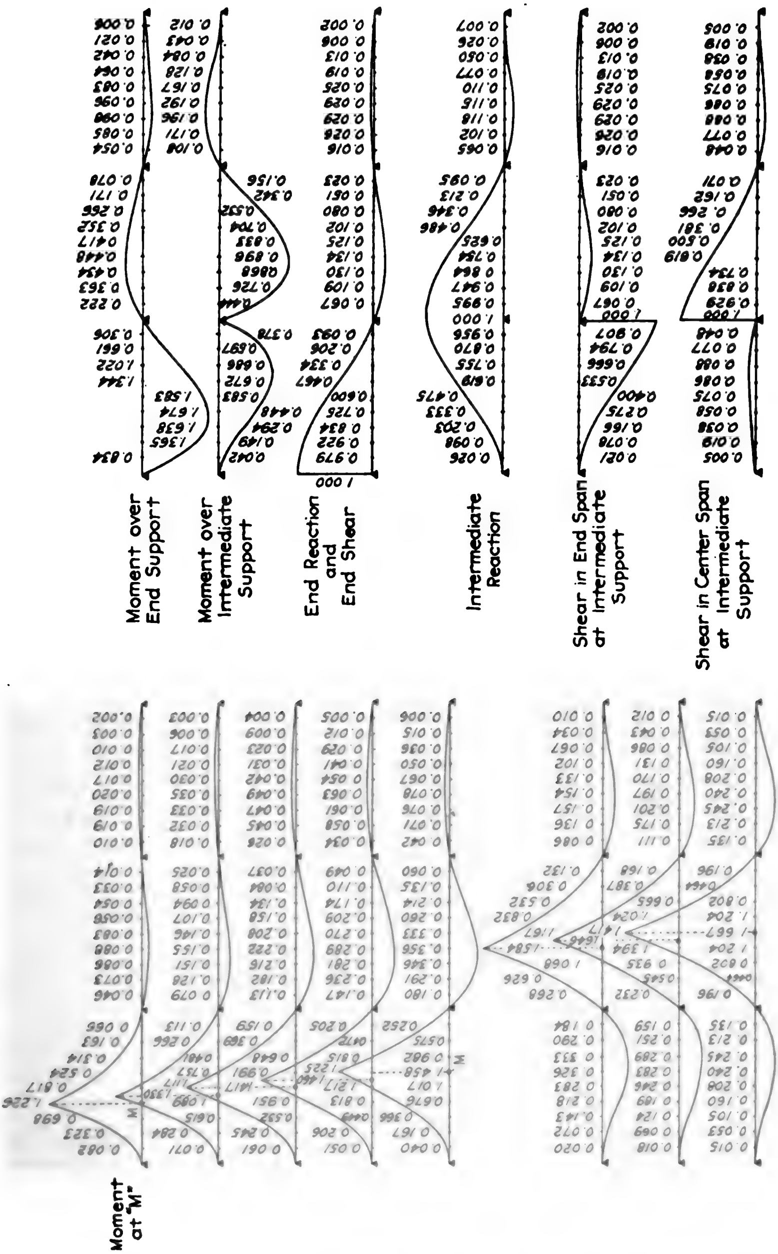


FIG. 293.—Influence lines for three equal spans, fixed ends. (Spans, 10 ft. Load, unity.)

In using influence lines for concentrated loads, the section where maximum positive moment occurs in any span of a continuous beam must be determined by trial. The sections chosen in plotting the influence lines will cover all ordinary cases and are sufficiently close together to give accurate enough results even if the true maximum occurs between these sections.

It should be clear to the student from "The Elements of Structures," just how the maximum values of the functions may be obtained by means of these influence lines when the spans are approximately equal and 10 ft. in length—both for fixed and moving concentrated loads, and load systems such as locomotive wheel loads on bridges. In the case of a load system, if all the loads are a whole number of feet apart, the ordinates given may be employed directly to determine the values of the maximum functions. If this is not the case it may be necessary to lay off a horizontal line to represent the length of the structure to a larger scale than that used in the text and then erect the ordinates at the given points. If the loads are then plotted on the edge of a separate strip of paper, the distances apart of the loads to the same scale as the length of the structure, then the strip can be moved along the structure and the successive values of the function determined by scaling to the influence line.

Suppose now the span length is other than 10 ft.—say 14 ft. Then it will be necessary to plot the distances apart of the loads only ten-fourteenths of the actual distance, and to the same scale as before. The influence lines—namely, length of base-lines and values of the ordinates—should not change. Now by moving the load strip as above plotted along the influence line, the true maximum value of the function under consideration may be determined. In case, however, the function is that of moment, the value as thus obtained should be multiplied by 1.4 in order to apply to the 14-ft. span.

Influence lines for continuous beams are not a series of straight lines as in the case of simply supported structures, but consist of arcs of curves. The value of a function should thus be computed for small changes in the position of the loads, but such computations need not be made for successive positions along the entire length of the structure. In each case there will arise limiting positions of the load beyond which it will be clear that no maximum can be obtained.

Problems

18. Assume a continuous beam of two 15-ft. spans and with fixed ends. Find the moment over the center support for 10,000-lb. loads at the quarter points in each span.
19. A continuous beam of three 15-ft. spans and supported ends is subject to a moving load of 25,000 lb. anywhere along its length. Find the maximum moment and maximum shear which may occur on this beam.
20. Solve Problem 19 assuming two moving loads of 25,000 lb. each, 5 ft. apart.

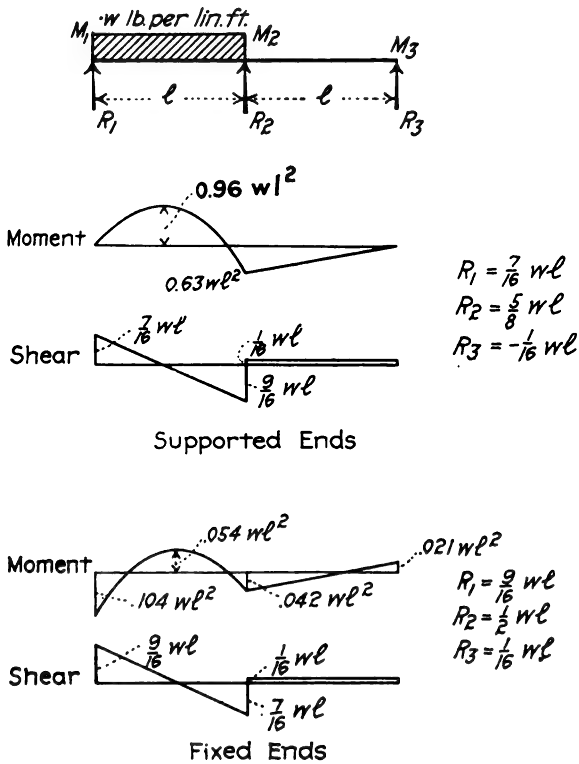


FIG. 294.—Moving uniform load, two equal spans.

Moving Uniform Loads

If a uniform load is considered, the influence lines indicate the spans which should be loaded in order to obtain the maximum

values of the given functions. Fig. 294 represents the variation in moment and shear for a uniform load on one span of a beam of two equal spans, both fixed and supported ends. Figs. 295, 296, and 297 give values for various loadings with three equal spans.

To illustrate the effect of loads on various spans upon the bending moments, influence lines have been drawn for six equal

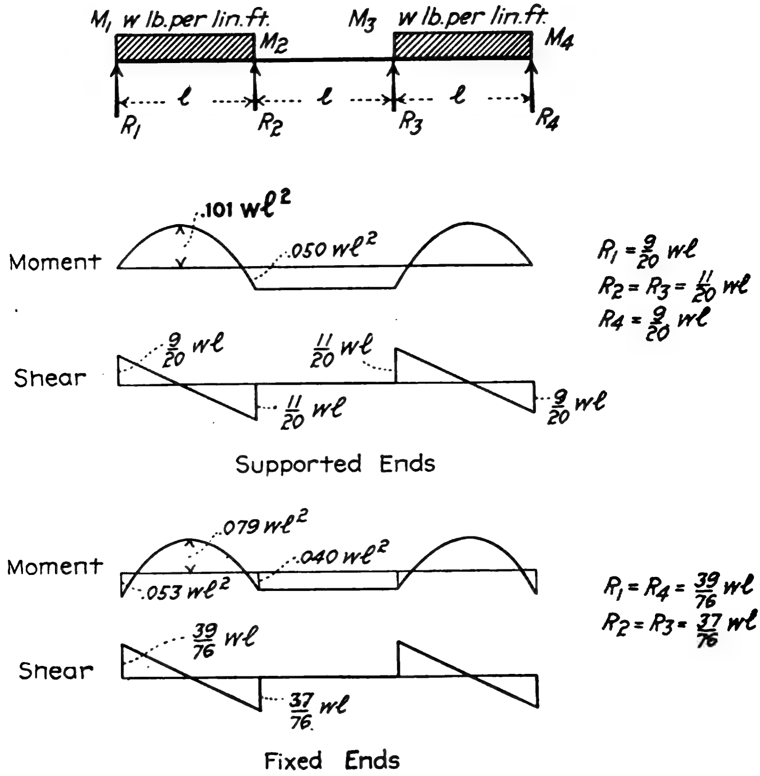


FIG. 295.—Moving uniform load, three equal spans.

spans (Fig. 298), for moments at the centers of span 1–2 and 3–4, and at supports 2 and 4. A maximum moment at the center of a span requires each alternate span to be loaded, and a maximum moment at the support requires the two adjacent spans to be loaded and then each alternate span. The effect of loads on remote spans is seen to be small. Influence lines are not drawn

for shears since, in a large number of spans, the shears do not differ greatly from those in simple beams.

Maximum Moments from Uniform Loads

If we consider now the maximum moments which may be obtained in continuous beams for uniform fixed and moving loads, it is quite evident from the table on page 373 (which explains

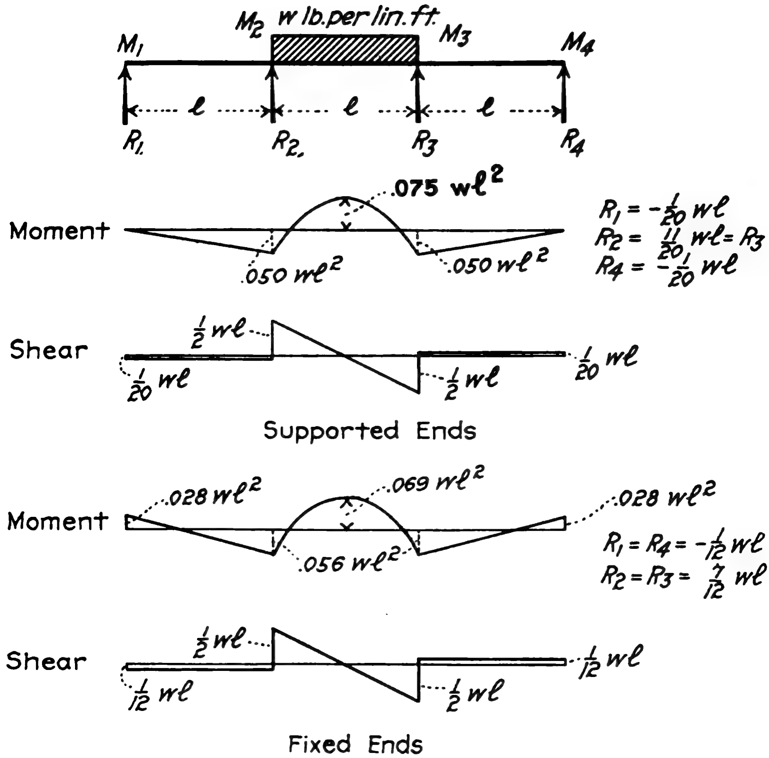


Fig. 296.—Moving uniform load, three equal spans.

itself) that the maximum positive and negative moments do not vary greatly in the several spans, with the exception of the end span and the adjoining support. For the immediate spans, the greatest value for the several spans is given in each case. Only beams with supported ends are considered in the table as it is

clear that these values control. This is entirely on the safe side since we know that the ends of continuous beams in reinforced-concrete building construction are fixed to some extent in nearly all cases. It should be noted that, for beams of five or more spans, the maximum moments are but little affected by the number of

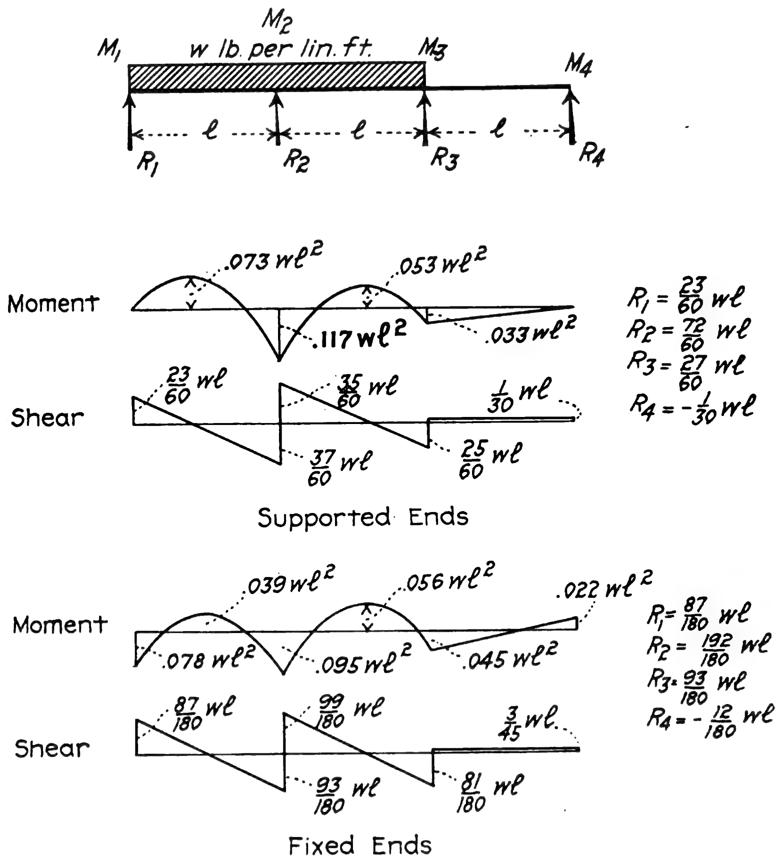


FIG. 297.—Moving uniform load, three equal spans.

spans. In the case of several spans, it has been assumed in calculating positive moments for many of the values of the table that the maximum moment at the center of the span is the maximum desired. This is sufficiently close to the truth. The two-span beam may well be treated as a special case.

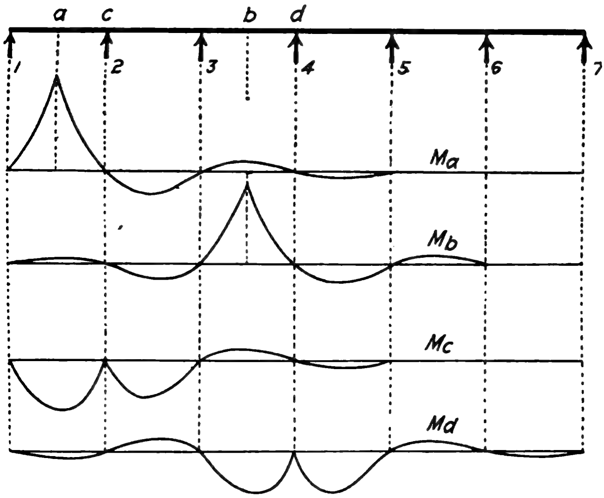


FIG. 298.

MAXIMUM MOMENTS IN CONTINUOUS BEAMS; SUPPORTED ENDS; UNIFORM FIXED AND MOVING LOADS

Coefficients of (wl^2)

No. of spans	Intermediate spans and supports				End span and second support			
	At center of span (+)		At support (−)		At center of span (+)		At support (−)	
	Fixed load	Moving load	Fixed load	Moving load	Fixed load	Moving load	Fixed load	Moving load
Two	.....	.....	.....	....	0.070	0.096	0.125	0.125
Three	0.025	0.075	.....	....	0.080	0.101	0.100	0.117
Four	0.036	0.081	0.071	0.107	0.077	0.098	0.107	0.120 (0.115)[1]
Five	0.046	0.086	0.079	0.111 (0.106)[1]	0.078	0.099	0.105	0.120 (0.116)[1]
Six	0.043	0.084	0.086	0.116 (0.106)[1]	0.078	0.099	0.106	0.120 (0.116)[1]
Seven	0.044	0.084	0.085	0.114 (0.106)[1]	0.078	0.099	0.106	0.120 (0.116)[1]

[1] Where two adjacent spans only are loaded.

The fixed load coefficients will apply to the dead load when finding the maximum coefficients due to a moving uniform load —the case we ordinarily have to deal with in building construction. Inasmuch as the theoretical maximum moments in continuous beams of five or more spans would involve unreasonable assumptions as to position of the live loads, the following values of moment coefficients may be taken:

Nature of load	Intermediate spans		End spans	
	At center	At support	At center	At support
Dead load	0.046	0.086	0.080	0.107
(two spans)			(0.070)	(0.125)
Live load	0.086	0.107	0.101	0.117
(two spans)			(0.096)	(0.125)

Combining the dead and live loads into a single unit for the purpose of determining general moment coefficients which will apply to all ordinary cases, we obtain the following:

Ratio of live to dead	Intermediate spans		End spans	
	At center	At support	At center	At support
Three or more spans				
2 : 1	0.073	0.100	0.094	0.114
5 : 1	0.079	0.104	0.098	0.115
10 : 1	0.082	0.105	0.099	0.116
Two spans				
2 : 1			0.087	0.125
5 : 1			0.092	0.125
10 : 1			0.093	0.125

In continuous-beam computations, the beam is assumed as freely supported at the interior supports and the assumption is made that the supports are of no appreciable width. For beams in concrete construction, therefore, the coefficients in the second column of the above table should be reduced, and could very

well be taken equal to those in the first column. The live load will generally range from two to five times the dead load, but the ratio of 10:1 is given to show the slight variation in moment coefficients for ratios above 5:1.

From a study of the table, reducing the bending-moment coefficients at the interior supports as above proposed, it is seen that the bending moment at the center and at the support for interior spans may be taken at $\frac{wl^2}{12}$ (0.083 wl^2), and for end spans it may be taken at $\frac{wl^2}{10}$ for center and adjoining support, where w includes both dead and live loads. In the case of two spans only, the bending moment at the center support may be taken at $\frac{wl^2}{8}$, and near the middle of the span at $\frac{wl^2}{10}$.

The shear at each support of continuous beams with fixed ends may be taken at one-half the span load. If the ends are simply supported, the shear in the end spans near the second support will be approximately 0.6 wl.

Beam Concentrations

Figs. 299 to 302 inclusive give the shears and moments caused by beam concentrations on continuous girders. The girder spans are assumed equal and the ends of the girders as simply supported. In girders with fixed ends and with full loading as shown in Fig. 299, the maximum positive moment for loads at the middle points is the same as the maximum negative moment and equals 0.125 Pl in every case. For loads at the third points, the maximum positive moment is 0.110 Pl and the maximum negative moment is exactly twice this value. For loads at the middle and quarter points, the maximum positive moment is 0.187 Pl and the maximum negative moment is 0.313 Pl. The maximum shears in all cases are the same as in simple beams.

In the following table are given the maximum positive and negative moments due to beam concentrations. Ends of beams are assumed as simply supported.

It is quite evident that the variation of moment coefficients is very nearly the same as shown in the table previously given for uniform loads.

MAXIMUM MOMENTS IN CONTINUOUS GIRDERS DUE TO BEAM CONCENTRATIONS; SUPPORTED ENDS

Coefficients of (Pl)

No. of spans	Intermediate spans and supports				End span and second support			
	At center of span (+)		At support (−)		At center of span (+)		At support (−)	
	Fixed load	Moving load	Fixed load	Moving load	Fixed load	Moving load	Fixed load	Moving load
Loads at middle points								
Two	......	......	......	......	0.156	0.203	0.187	0.187
Three	0.100	0.175	......	......	0.175	0.213	0.150	0.175
Five	0.130	0.191	0.119	0.156[1]	0.171	0.211	0.158	0.174[1]
Loads at third points								
Two	......	......	......	......	0.222	0.278	0.333	0.333
Three	0.066	0.200	......	......	0.244	0.289	0.267	0.311
Five	0.122	0.228	0.211	0.276[1]	0.240	0.286	0.281	0.309[1]
Loads at middle and quarter points								
Two	......	......	......	......	0.267	0.383	0.465	0.465
Three	0.128	0.312	......	......	0.314	0.406	0.372	0.438
Five	0.204	0.352	0.296	0.389[1]	0.303	0.401	0.394	0.435[1]

By similar reasoning to that employed in deriving moment coefficients for uniform loads, we obtain the following values:

Nature of load	Intermediate spans		End spans	
	At center	At support	At center	At support
Loads at middle points				
Dead load	0.130	0.119	0.175	0.158
(two spans)	......	......	(0.156)	(0.187)
Live load	0.191	0.156	0.213	0.175
(two spans)	......	......	(0.203)	(0.187)
Loads at third points				
Dead load	0.122	0.211	0.244	0.281
(two spans)	......	......	(0.222)	(0.333)
Live load	0.228	0.276	0.289	0.311
(two spans)	......	......	(0.278)	(0.333)
Loads at middle and quarter points				
Dead load	0.204	0.296	0.314	0.394
(two spans)	......	......	(0.267)	(0.465)
Live load	0.352	0.389	0.406	0.438
(two spans)	......	......	(0.383)	(0.465)

[1] Two adjacent spans only are loaded.

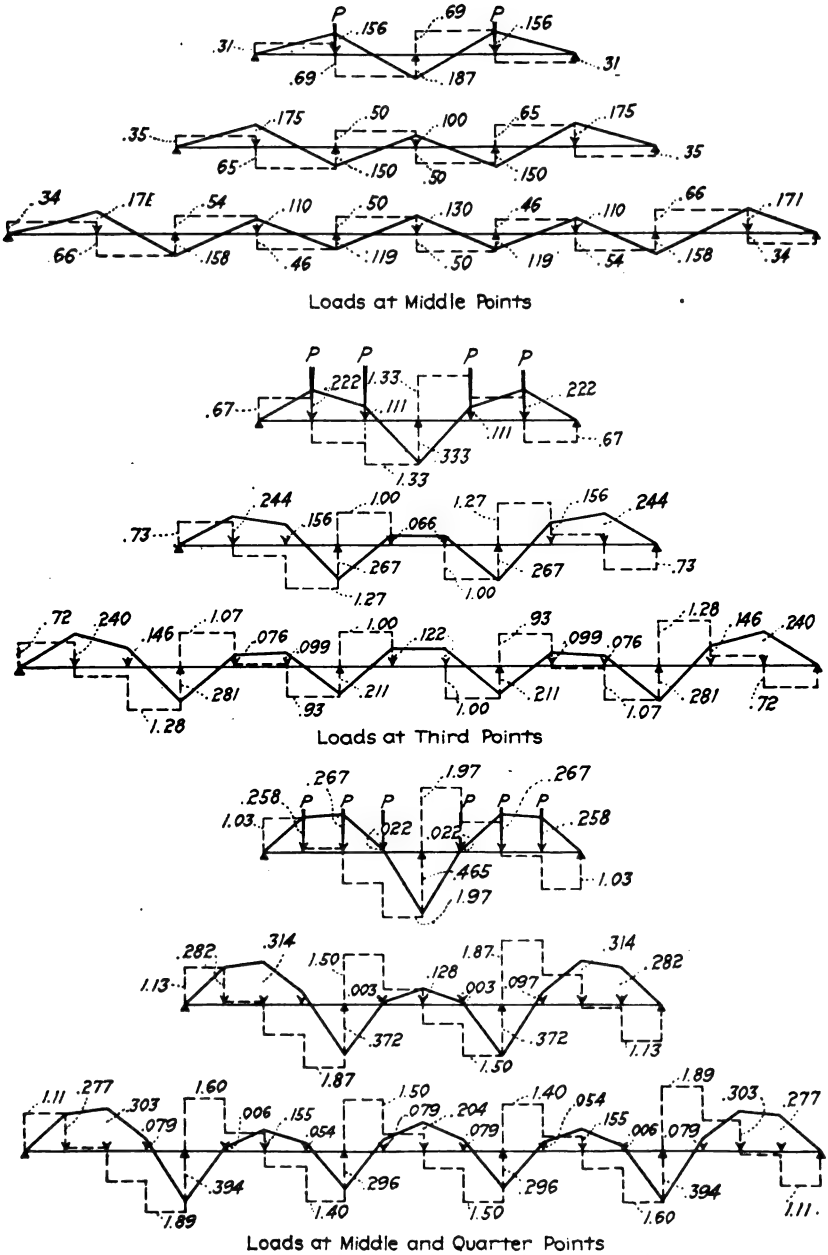


Fig. 299.—Moments and shears in continuous beams; supported ends; spans all equal; concentrated loads as shown. Coefficients of (Pl) for moment. Coefficients of (P) for shear.

Combining the dead and live loads into a single unit for the purpose of determining general moment coefficients, we obtain the following:

Ratio of live to dead	Intermediate spans		End spans	
	At center	At support	At center	At support
Loads at middle points				
2 : 1	0.171	0.143	0.200	0.169
5 : 1	0.181	0.150	0.207	0.172
(Two spans)				
2 : 1	....	....	0.187	0.187
5 : 1	....	....	0.195	0.187
Loads at third points				
2 : 1	0.193	0.254	0.274	0.301
5 : 1	0.210	0.265	0.281	0.306
(Two spans)				
2 : 1	....	....	0.259	0.333
5 : 1	....	....	0.269	0.333
Loads at middle and quarter points				
2 : 1	0.303	0.358	0.375	0.423
5 : 1	0.327	0.374	0.391	0.431
(Two spans)				
2 : 1	....	....	0.344	0.465
5 : 1	....	....	0.364	0.465

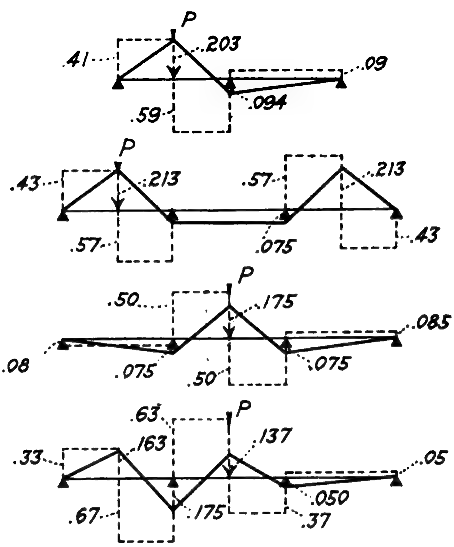


FIG. 300.—Concentrated loads as shown; loads at middle points; two and three equal spans; supported ends. Coefficients of (Pl) for moment. Coefficients of (P) for shear.

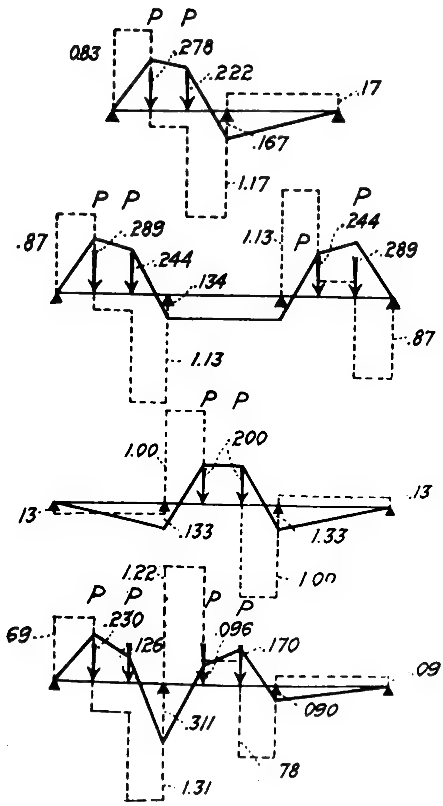


FIG. 301.—Concentrated loads as shown; loads at third points; two and three equal spans; supported ends. Coefficients of (Pl) for moment. Coefficients of (P) for shear.

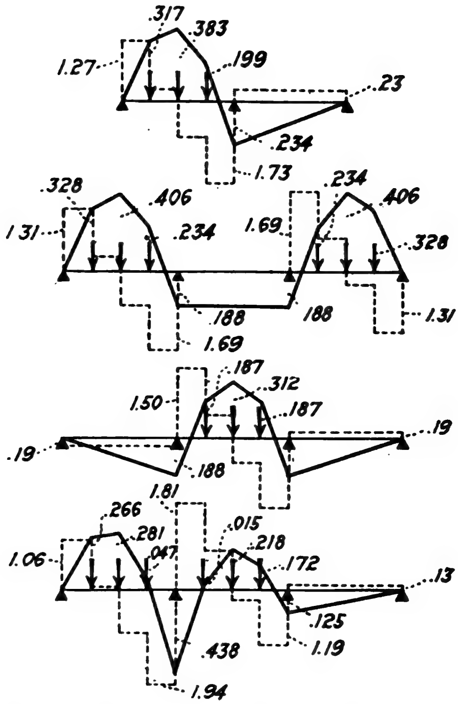


FIG. 302.—Concentrated loads as shown; loads at middle and quarter points; two and three equal spans; supported ends. Coefficients of (Pl) for moment. Coefficients of (P) for shear.

In Art. 54 of Volume I, the statement was made that for occasional concentrated loads which act in connection with uniform live and dead loads, and for loads produced by beams running into girders, the maximum moment may be computed with sufficient accuracy by considering the beam or girder as simply supported, and then reducing this maximum moment (and an equal negative moment) by the same ratio used in the distributed loading. This rule gives moment coefficients as follows:

No. of spans	Intermediate spans	End spans and adjoining supports
Loads at middle points		
Three or more spans	0.167	0.208
Two spans	0.208	0.250
Loads at third points		
Three or more spans	0.222	0.278
Two spans	0.278	0.333
Loads at middle and quarter points		
Three or more spans	0.333	0.417
Two spans	0.417	0.500

In a beam loaded at the middle points we find that the moment at the center of intermediate spans may have a value about 8 per cent greater than the recommended value for use in des'gn. (This, however, is considering the beam as freely supported at the interior supports, and the supports of no appreciable width.) All other values for this loading are somewhat less than those recommended. In fact, the specified moment coefficient for the center support of a two-span beam may be reduced and made the same as for the center of span. In beams of three or more spans and for the same loading as just mentioned, the moment coefficient for the inner supports of end spans may be reduced so as to have the same value as specified for the interior spans.

Now let us consider the moments in beams loaded at the third points. The moment at interior supports may have a value about 19 per cent greater than that specified, but the width and monolithic character of the supports will offset this to a considerable extent. It is preferable, however, to make some allowance for this in design although for simplicity this has not been done in this text. The same is true for the inner sup-

ports of the end spans although the increase in moment over that specified is about 10 per cent.

The student may draw his own conclusions in regard to moment coefficients in beams loaded at the middle and quarter points.

The recommendations for shear previously given for uniform loads will apply in the case of beam concentrations.

Triangular Distribution of Load

In Art. 56 of Volume I it was shown that where floor slabs are reinforced in two directions, the loads carried to the cross-beams and girders should be assumed to vary in accordance with the ordinates of a triangle. The statement was also made that the maximum moment on the cross-beams and girders may be computed with sufficient accuracy by considering the beam or girder as simply supported, and then reducing this maximum moment (and an equal negative moment) by the same ratio used in the uniformly distributed loading. The theorem of three moments for this case takes the following form:

$$M_2l_2+M_3(l_2+l_3)+M_4l_3=-\frac{5}{24}W_1l_1^2-\frac{5}{24}W_2l_2^2$$

where W_1 = total load on left span.
W_2 = total load on right span.

The same method might be followed here in showing the reason for the above rule as was employed for uniform loading and for beam concentrations, but the student by this time should understand the method of procedure.

Negative Moment at the Ends of Continuous Beams

The amount of negative moment at the ends of continuous beams depends upon the manner in which the ends are restrained. A beam cannot be entirely fixed unless the restraint is sufficient to cause the neutral surface at the ends to be horizontal. (See Art. 53 of Volume I and the Joint Committee's report at the end of this volume.) The moment coefficient must be left to the judgment of the designer, but the shear and moment diagrams shown in Figs. 294 to 297 inclusive (for beams with fixed ends) and in Figs. 310 and 312 will prove useful in this connection.

INCREASE OF POSITIVE MOMENTS IN CONTINUOUS BEAMS DUE TO UNEQUAL SETTLEMENTS OF THE SUPPORTS

Unequal settlement of supports may be caused by faulty foundations or by the deflection of a girder carrying the ends of one or more cross-beams. The first condition will not be considered since it is the result of poor design in the foundations. As regards the second condition, consideration will show that a relative settlement of about $\frac{1}{12}$ in. may be expected under usual conditions. If a cross-beam of two spans is considered supported at the center by a girder in which a deflection of $\frac{1}{12}$ in. occurs, the negative moment in the cross-beam is reduced but the positive moment will be about 4 per cent. greater than the moment determined for supports on a level. This gives some idea of the increase in moment which is likely to occur due to unequal settlement of supports.

BENDING UP OF BARS AND PROVISION FOR NEGATIVE MOMENT

In Figs. 303 to 305 inclusive are given bending-moment curves for uniform loads on two, three and four equal spans. The curves for live load are not bending-moment curves for any single arrangement of loading. In fact, for each curve several

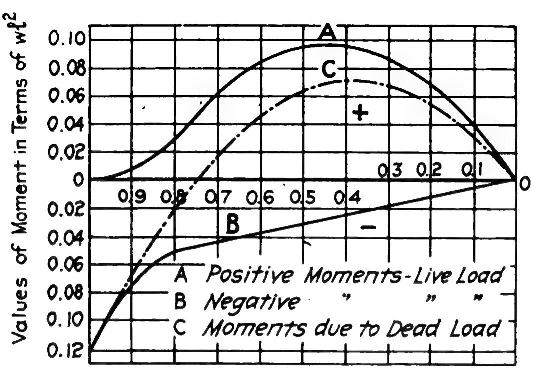


FIG. 303.—Moment curves for uniform load; two spans; right-hand half; supported ends.

conditions of loading are considered and the curve is obtained by plotting maximum or minimum values as the case may be. It is necessary to plot the curves for only one-half the beam, as the center line shown is on the plane of symmetry.

The student will notice that some portions of the live-load curves are quite different from those shown in Figs. 287 and 294 to 297 inclusive. For example, the part of the maximum live-load curve close to the center support in the beam of two spans

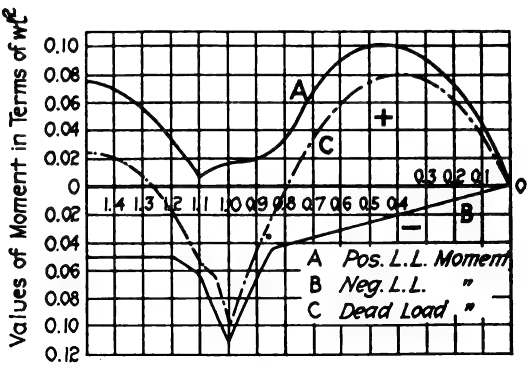


FIG. 304.—Moment curves for uniform load; three spans; right-hand half; supported ends.

(Fig. 303) is quite different from a similar portion in any of the curves shown in Figs. 287 and 294. This is due to the fact that for these sections maximum and minimum moments are caused by only partial loading, and not by having either one or both

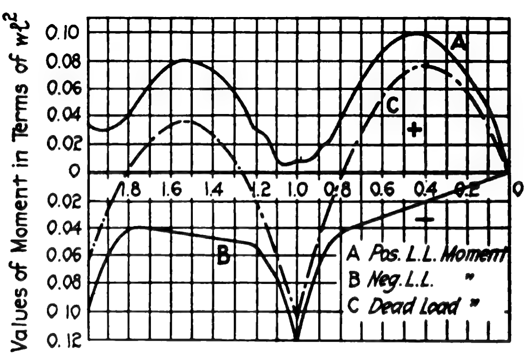


FIG. 305.—Moment curves for uniform load; four spans; right-hand half; supported ends.

spans fully loaded. If influence lines were plotted for these sections, this point would be clearly brought out.

In the two-span beam shown in Fig. 306, maximum and mini-

mum bending-moment curves are given for a 2 : 1 ratio of live to dead load. To obtain these curves from Fig. 303, points should be determined for each one-tenth of the span. The following notation will be used:

w_l = live load per unit length.
w_d = dead load per unit length.
$w_t = w_l + w_d$ = total load per unit length.

Referring to Fig. 303, consider a point at a distance $0.4l$ from the freely-supported end of the right-hand span. The live-load moment is seen to vary between $+0.095w_ll^2$ and $-0.025w_ll^2$.

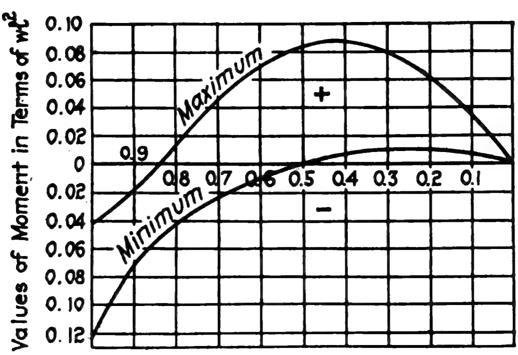


FIG. 306.—Moment curves for continuous beams of two spans; supported ends. Ratio of live load to dead load 2 : 1.

The first of these corresponds to the condition of loading in which the right-hand span alone is loaded, and the second value corresponds to the left-hand span being loaded. In addition to these moments, there is that due to the dead load, which the curve shows to be $0.07w_dl^2$ for the point in question.

Combining these moments, we see that the moment may vary between

$$0.07w_dl^2 + 0.095w_ll^2$$

and

$$0.07w_dl^2 - 0.025w_ll^2$$

Assuming $w_l = 2w_d$, the moment varies from $+0.26w_dl^2$ to $+0.02w_dl^2$; that is (since $w_d = \frac{1}{3}w_t$), from $+0.0867w_tl^2$ to $+0.0067w_tl^2$.

Curves such as shown in Fig. 306 show [illegible] moment

of resistance should be provided in either direction at any point along a two-span beam with supported ends and with $w_l = 2w_d$. They also indicate what proportion of the steel may be bent up from the lower side at any point, without exceeding the safe

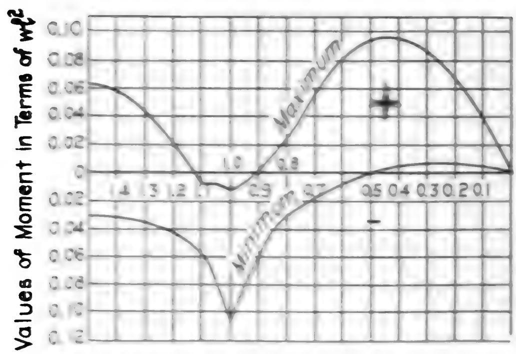


FIG. 307.—Moment curves for continuous beams of three spans; supported ends. Ratio of live load to dead load 3 : 1.

stress on the remaining rods. The curves show that a negative moment is likely to occur over one-half of each span. Any fixing of the ends, however, will reduce this length.

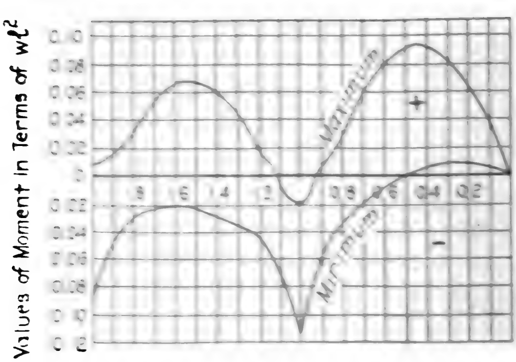


FIG. 308.—Moment curves for continuous beams of four spans; supported ends. Ratio of live load to dead load 3 : 1.

Referring to Figs. 306, 307, and 308, it is clear that negative moment may, under extreme conditions, occur entirely across the beam. For ordinary cases, then, it would seem that top reinforcement should extend to at least the third point. In special

cases, however, it may be desirable to provide for negative-tension reinforcement over the entire span. If reinforcing frames are used, such as shown in Fig. 66, the top rods employed for handling and for fastening the stirrups into a unit will aid materially in taking care of any tensile stresses which may occur in the top of the beam. It is also true that in monolithic floor

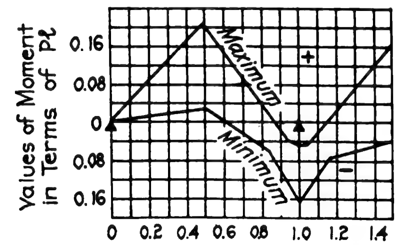


FIG. 309.—Moment curves for continuous beams of three spans; supported ends; loads at middle points. Ratio of live load to dead load 4 : 1.

construction, the adjoining slab will help considerably in preventing top tensile stresses at the center of the span.

In view of the above considerations, it would seem that rods may be bent up with sufficient accuracy (for ordinary cases where uniform live load is somewhat indefinite) by applying the formula

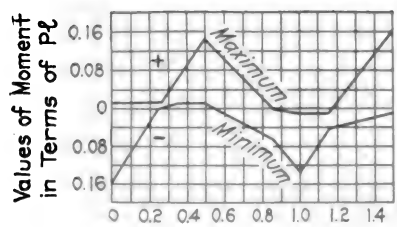


FIG. 310.—Moment curves for continuous beams of three spans; fixed ends; loads at middle points. Ratio of live load to dead load 4 : 1.

of Art. 63, Volume I. For special cases, curves should be drawn similar to those shown in Figs. 306 to 308 inclusive.

Maximum and minimum moment curves for concentrated loads are shown in Figs. 309 to 313 inclusive. Three-span beams only are considered and curves are given for both supported and fixed ends. From a study of these curves and in view of the considerations presented previously in this article, it would seem

(assuming the usual indefinite live load) that rods in girders loaded at the third points may safely be bent up as explained in the floor-bay design of Art. 15. The curves are given for a 4:1 ratio of live to dead load. The moment due to dead weight of

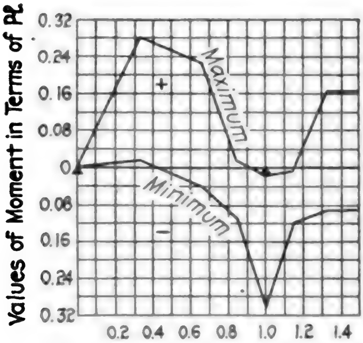


Fig. 311.—Moment curves for continuous beams of three spans; supported ends; loads at third points. Ratio of live load to dead load 4:1.

girder stem has little effect in considerations regarding the bending up of rods.

The formulas given below may be of use when considering the variation of moment in beams of *many* equal spans for different kinds of loading. In deriving the formulas for maximum positive

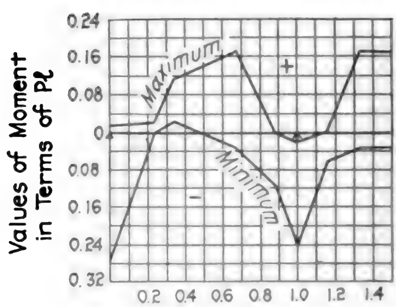


Fig. 312.—Moment curves for continuous beams of three spans, fixed ends; loads at third points. Ratio of live load to dead load 4:1.

moment, alternate spans were considered as covered with live load. This gives the worst condition of loading for positive moment. Likewise in deriving the formulas for maximum negative moment, the worst condition of loading for negative moment

was assumed—that is, two adjacent spans were assumed as fully loaded alternating with one span unloaded. For the same load on each span, the formulas reduce to simple terms, and give the moments on continuous beams with fixed ends and with any number of spans.

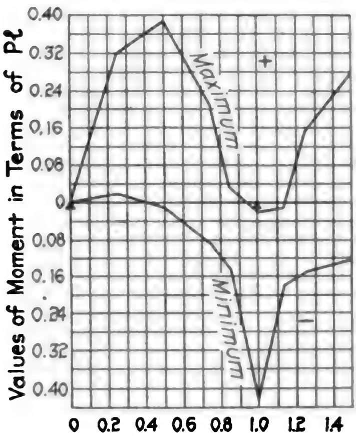


FIG. 313.—Moment curves for continuous beams of three spans, supported ends; loads at middle and quarter points. Ratio of live load to dead load 4 : 1.

The following notation will be employed:

$M_{max.\ p.}$ = maximum positive moment at center of span.

$M_{min.\ p.}$ = minimum positive moment (or maximum negative moment) at center of span.

$M_{max.\ n.}$ = maximum negative moment at the support.

Uniform loads:

$$M_{max.\ p.} = \frac{w_l l^2}{12} - \frac{w_d l^2}{24}$$

$$M_{min.\ p.} = \frac{w_d l^2}{12} - \frac{w_l l^2}{24}$$

$$M_{max.\ n.} = -\frac{w_l l^2}{9} + \frac{w_d l^2}{36}$$

Loads at middle points:

$$M_{max.\ p.} = \frac{l}{16}(3P_l - P_d)$$

$$M_{min.\ p.}=\frac{l}{16}(3P_d-P_t)$$

$$M_{max.\ n.}=-\frac{P_tl}{6}+\frac{P_dl}{24}$$

Loads at third points:

$$M_{max.\ p.}=\frac{l}{9}(2P_t-P_d)$$

$$M_{min.\ p.}=\frac{l}{9}(2P_d-P_t)$$

$$M_{max.\ n.}=-\frac{8P_tl}{27}+\frac{2P_dl}{27}$$

Triangular distribution of load:

$$M_{max.\ p.}=\frac{l}{96}(11W_t-5W_d)$$

$$M_{min.\ p.}=\frac{l}{96}(11W_d-5W_t)$$

$$M_{max.\ n.}=-\frac{5}{36}W_tl+\frac{5}{144}W_dl$$

69. Beams with Varying Moment of Inertia.—It is the practice of some designers to place more steel between the supports of continuous beams than the amount just sufficient to resist the bending moment specified by the Joint Committee. They consider that by doing this the stresses over the supports are reduced and the design is more economical. They maintain, also, that this method of procedure is advisable in order to provide for imperfect continuity of the beam and to take care of unknown stresses caused by unequal settlements of the supports. It is important, therefore, to determine the actual moments which occur at the center and supports in such cases.

A study of this kind, assuming uniform loads, has been made by Mr. R. E. Spaulding[1] for beams with fixed ends and for beams with one end fixed and one end free. The curves shown in Fig. 314 are the result of his investigation and apply to either rectangular or T-beams. Mr. Spaulding assumed a uniform moment of inertia for that portion of the beam over the support in which negative moments exist, and another moment of inertia

[1] *Engineering News*, Jan. 13, 1910.

for the central portion of the span—that is, between points of inflection.

The condition of fixed ends is obtained for full loading on the lower floors of a building having very heavy columns. The case of one end fixed and one end free is found in either span of a two-span beam with the same uniform load on both spans.

Referring to the curves of Fig. 314 it is clear that with both ends fixed and with the same moment of inertia throughout, the moment at the center is $0.042wl^2 = \frac{wl^2}{24}$ and at the end is $0.083wl^2 = \frac{wl^2}{12}$, as is well known. If, as is sometimes done, a

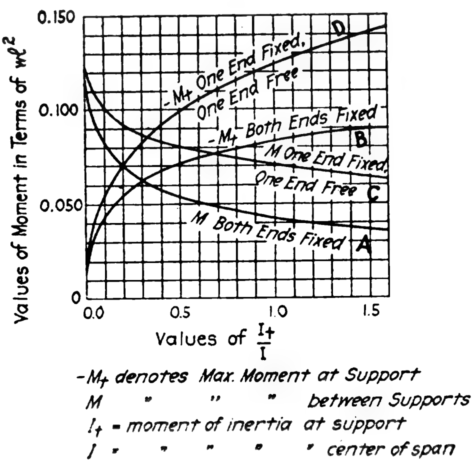


FIG. 314.—Curves of maximum bending moments in beams with different moments of inertia at end and centers.

small amount of steel is placed over the supports, such that $\frac{I_t}{I} =$ 0.20, and the full bending moment $\frac{(wl^2)}{8}$ is provided for at the center, the actual moment at the center for fixed ends will be 0.069 wl^2 (55 per cent of $\frac{wl^2}{8}$) and at the support will be 0.056 wl^2. Such a condition stresses the steel at the support to a value 2.2 times the working stress since provision is made at the support for a moment of only $(0.20)\ (\frac{1}{8}wl^2) = \frac{1}{40}wl^2 = 0.025wl^2$— that is, of course, assuming the amount of steel used to be proportional to the moments of inertia. Again, suppose the

amount of steel at the support of a beam with fixed ends to be made one-half that at the center and that the center be designed for $\frac{1}{10}wl^2$. The actual moment at the center will be 0.053 wl^2 and the end bending moment will be $0.073wl^2$. Thus, with this distribution of the reinforcement, the steel at the support will be overstressed to an amount $\frac{0.073}{0.050}=1.5$ times the assumed working stress. It may be seen from the curves of Fig. 314 (for beams with fixed ends) that if a beam is figured as simply supported, then about six-tenths of the amount of steel that is employed in the middle of the beam should be placed over supports in order to make as economical a design as possible under the conditions. For example, with this distribution of steel, provision is made at the support for a moment of

$$0.6\left(\frac{wl^2}{8}\right)=0.075\ wl^2$$

and this is exactly the actual moment caused by such an arrangement. The assumption which is made that the moments of inertia are proportional to the amount of steel used makes practically no difference in these comparative results. The student can verify this statement by working out a number of designs considering either rectangular or T-beams.

In the above discussion no compressive steel has been considered at the supports. In rectangular beams, of course, there is none needed but in T-beams the section becomes rectangular at the supports and the stress in the concrete needs attention. Referring to Table 3 of Volume I, it may be seen that for an allowable compressive stress in the concrete at the support of 750 lb. per square inch and 16,000 lb. in the steel, the required percentage of tensile steel is practically 1 per cent. It is thus possible to get along without any additional strengthening of the compressive part of the beam at the support only when the beam is reinforced at the center of span with less than 1 per cent $\div$ 0.6 = 1.7 per cent of the area of the stem (including the portion of the slab directly above it), whether or not 1.7 per cent is less than the amount determined for the full bending moment of $\frac{wl^2}{8}$.

The question naturally arises whether it is more economical to design for the full bending moment of $\frac{wl^2}{8}$ and provide steel over supports equal in amount to six-tenths of the steel at the middle of beam, or, as recommended by the Joint Committee, to design

for $\frac{wl^2}{12}$ both at the center of span and over supports. Consider a given case where a continuous beam designed as simply supported requires 3 sq. in. of steel at the center of span. Of this amount (0.6) (3) = 1.8 sq. in. should be bent up and carried over the top of the support (disregarding amount of steel required for bond), the rest of the rods being horizontal in the bottom of the beam. The approximate volume of this steel may be taken as the area of the steel in the middle times the span in inches, or 3 sq. in. $\times l = 3l$ cu. in. The same beam designed according to the Joint Committee's recommendation would require 2 sq. in. of steel at the center of span and 2 sq. in. over the top at the support, and the volume of steel may be taken as the volume of the long rods, the area of which amounts to 2 sq. in., plus the volume of steel carried over from the adjoining spans and extending to about the one-third point of the span, or approximately (2 sq. in.) $(l) + (2)(\frac{1}{3}l) = 2\frac{2}{3}l$ cu. in. The student should notice in the above comparison that the first design is favored to the extent that no top steel is assumed to extend beyond the center of the support. Assume this steel to extend to only the one-fifth point of the span, then the volume of steel in the first case becomes equal to $3l + (1.8)(\frac{1}{5}l) = 3.4$ sq. in. Thus it is clearly seen that a beam designed for a moment $\frac{wl^2}{8}$ in the center of span requires approximately 25 per cent more steel than the beam designed with $\frac{wl^2}{12}$, for both positive and negative moments.

Mr. Spaulding in his study of continuous beams (referred to above) assumed that the moment of inertia is constant between the points of inflection, and there changes abruptly to another value which is constant for the ends of the beam. This assumption neglects in all cases the value of concrete below the neutral axis because it is in tension and is liable to crack. This is true, however, only for sections subjected to the maximum bending moments. In fact, the variation of the moment of inertia throughout the beam may be represented by a curve with maximums at the points of inflection and minimums at the middle of the beam and at the supports. Mr. Sanford E. Thompson in an article published in the issue of *Engineering News* of January 13, 1910, under the title "Continuity in Reinforced-concrete Beams." states that extensive studies made in his office, considering the

moment of inertia to vary in this way, gave results which substantially agree with those obtained by Mr. Spaulding and prove the latter's assertion that the assumption of constant moment of inertia between points of inflection is sufficiently accurate for all practical purposes.

Near the beginning of this article it is shown that a beam reinforced for the full bending moment of $\frac{wl^2}{8}$ at the center of span will induce a bending moment (if properly designed) of approximately $0.075wl^2$ over supports. Now this is only about 10 per cent less than the moment of $\frac{wl^2}{12}$ recommended by the Joint Committee, and it is almost self-evident from what has preceded and from the knowledge derived from Art. 68 as to the effect of unloaded panels, that no more economical and satisfactory arrangement of steel can be obtained for ordinary cases than that which the Joint Committee has specified.

Problems

21. Construct moment and shear diagrams for a beam of three spans loaded on only one of the outer spans. Consider both fixed and supported ends.
22. If the center of span of a continuous beam with practically fixed ends is reinforced for a moment $\frac{wl^2}{10}$, what is the least amount of steel that can be safely placed over the top of beam at the supports?

CHAPTER XIV

ECCENTRIC-LOAD CONSIDERATIONS IN COLUMNS

70. Considerations in General.—By means of the principles which were employed in the derivation of the theorem of three moments, it is possible to obtain formulas whereby the moments in any given span of a continuous beam may be determined if the slope of the neutral surface of the beam is known at each of the two adjoining supports. This is possible whether or not there is any load in the span in question. For no load in the span we have the case of an ordinary building column extending through one story height and eccentrically loaded—that is, by means of the formulas mentioned, it is possible to determine the moments in a column in ordinary building construction due to unsymmetrical floor loading if the slopes at the ends of the column are known. In the case of a column resting upon a footing of considerable size and monolithic with it, the column may usually be considered as fixed in direction at its lower end, and hence the neutral surface of the column at this point is vertical. At the first floor level, however (assuming there is a basement), the beam has a slope at the column in the case of unsymmetrical loading and this slope, of course, is impressed upon the upper end of the column.

Except where the load rests upon column brackets and has what might be called an intentional eccentricity, the method of finding the stress due to bending is to calculate the slope of the beam at its junction to the column and from this determine the bending moment acting on the column itself. With the bending moment known, the stress in the column may be found by means of the general formulas of Art. 74, Volume I. When the column section is rectangular with the steel placed as in a double-reinforced beam and symmetrical about the gravity axis, then the formulas of Arts. 75 and 76 of Volume I may be employed. Diagrams 13, 14, and 15 apply when the steel is imbedded in the concrete one-tenth of the total effective depth from each surface and when n is taken at 15. If desired, the eccentricity of loading may be found by dividing the bending moment by the column reaction.

Columns in ordinary construction are usually designed for full live load on ajdacent floor bays and no attention is paid to eccentricity of loading except in the case of exterior columns where an arbitrary amount of steel is provided to take care of the stress due to bending. This method is undoubtedly satisfactory for the usual case of an indefinite live load, but more exact calculations should be made in special cases and where the loading is more definitely known.

From what has been said in regard to the method of figuring columns for eccentric loading, it should be clear that we need to find the slope of the beam at each end of the column and then find the maximum moment in the column which would be caused by these slopes. In the upper stories of a building with several floors the stiffness of the columns do not greatly affect the slope which the beams take up under various conditions of loading, but in the lower stories the columns are usually so large and so heavily reinforced that it is desirable to take their stiffness into account. It should be understood, however, that to neglect column stiffness is always on the side of safety.

The formulas which will be given (and other more general formulas) are all derived in the appendix to the book on "Reinforced Concrete Design" by Faber and Bowie.[1] Students trained in the Calculus would do well to study through some of these proofs in order to be able to derive formulas for other cases.

71. Formulas.—The following notation will be used:

B = ratio of moment of inertia to length for a beam.

C = ratio of moment of inertia to length for a column.

α = the slope or angle made by the neutral surface at the end of a member with its original position.

E = modulus of elasticity.

K = constant in equation $M = KCE\alpha$

I_b = moment of inertia of beam.

I_c = moment of inertia of column.

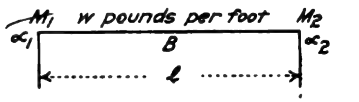
The slope of the neutral surface will be considered positive when the member has turned through a positive angle from its neutral position—that is, when it has turned in a counter-clockwise direction. It will be assumed in all cases that the moment of inertia of the beams is constant throughout any particular span. The formulas, therefore, apply without sensible error when approximately the same amount of steel is used over supports as

[1] Longmans, Green & Co., Publishers.

at the center of span. The same value of E will be taken for both beams and columns.

Beam Formulas

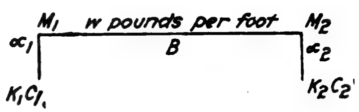
One span, uniformly loaded. Slopes at the ends of the beam are known.



$$M_1 = -\frac{wl^2}{12} - 4EB\alpha_1 - 2EB\alpha_2$$

$$M_2 = -\frac{wl^2}{12} + 2EB\alpha_1 + 4EB\alpha_2$$

One span, uniformly loaded, monolithic with columns.



$$\alpha_1 = -\frac{wl^2}{12E} \cdot \frac{(K_2C_2+6B)}{(K_1C_1+4B)(K_2C_2+4B)-4B^2}$$

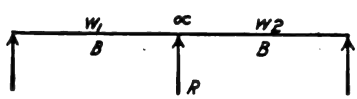
When, as is generally the case, $K_1C_1 = K_2C_2$,

$$\alpha_1 = -\frac{wl^2}{12E} \cdot \frac{1}{(KC+2B)}$$

When the stiffness of the columns is neglected,

$$\alpha_1 = -\frac{wl^2}{24BE}$$

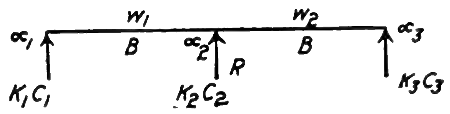
Two equal spans, freely supported, each with its uniform load.



$$\alpha = \frac{(w_1 - w_2)l^2}{48EB}$$

$$R = \frac{5(w_1+w_2)l}{8}$$

Two equal spans, monolithic with the columns, each with its uniform load.



When the two outside columns are similar, we have $K_3C_3 = K_1C_1$, and

$$\alpha_1 = -$$

$$\frac{l^2}{12E}\left[\frac{w_1\{(K_1C_1+4B)(K_2C_2+10B)-4B^2\}-w_2\{2B(K_1C_1+6B)\}}{(K_1C_1+4B)\{(K_1C_1+4B)(K_2C_2+8B)-8B^2\}}\right]$$

$$\alpha_2 = \frac{l^2}{12E}\left[\frac{(w_1-w_2)(K_1C_1+6B)}{(K_1C_1+4B)(K_2C_2+8B)-8B^2}\right]$$

When the beam is freely supported at the ends, then K_1C_1 and K_3C_3 equal zero, and

$$\alpha_2 = \frac{(w_1-w_2)l^2}{8E(K_2C_2+6B)}$$

$$R = \frac{5(w_1+w_2)l}{8}$$ (same as for beam freely supported throughout)

When the structure is completely monolithic but the stiffness of the columns is neglected,

$$\alpha_1 = -\frac{l^2}{12E}\cdot\frac{3w_1-w_2}{8B}$$

When w_2 is considered zero,

$$\alpha_1 = -\frac{wl^2}{32BE}$$

Three equal spans, monolithic with the columns, each with its uniform load, the beams and columns being symmetrical about the center line of the center beam.

When $B_1 = B_2$ and $K_2C_2 = K_1C_1$, we have

$$\propto_1 = -\frac{l^2}{12E}\left[\frac{w_1(KC+8B)-2w_2B}{K^2C^2+10KCB+20B^2}\right]$$

When $B_1 = B_2$ and $K_2C_2 = 2K_1C_1$, we have

$$\propto_1 = -\frac{l^2}{12E}\left[\frac{w_1(KC+4B)-w_2B}{(KC+2B)(KC+5B)}\right]$$

When the beam is freely supported at the ends, and $B_1 = B_2$,

$$\propto_2 = \frac{l^2}{12E}\cdot\frac{(1.5w_1-w_2)}{(KC+5B)}$$

$$R = 0.55l(w_1+w_2) \text{ (approximately)}$$

When the beam is freely supported at the ends, and $B_1 = 1.25B_2$,

$$\propto_2 = \frac{l^2}{12E}\cdot\frac{(1.5w_1-w_2)}{(KC+5.75B)}$$

$$R = 0.55\, l(w_1+w_2 \text{ (approximately)}$$

In general, for ordinary values of $\frac{B_2}{B_1}$,

$$\propto_2 = \frac{(3w_1-2w_2)}{\left\{10+7.5\left(1-\frac{B_2}{B_1}\right)\right\}B_2}\cdot\frac{l^2}{12E} \text{ (approximately)}$$

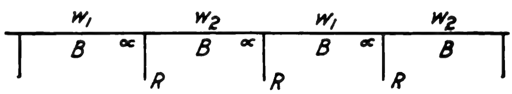
When the structure is completely monolithic but the stiffness of columns is neglected,

$$\propto_1 = -\frac{l_2}{12E}\cdot\frac{4w_1-w_2}{10B}$$

When w_2 is considered zero,

$$\propto_1 = -\frac{wl^2}{30BE}$$

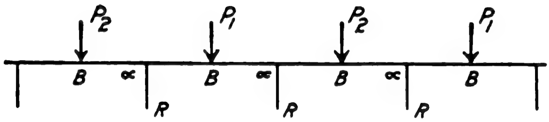
Beams of many equal spans, uniformly live loaded on alternate bays, and monolithic with the columns.



$$\propto = -\frac{l^2}{12E}\cdot\frac{(w_1-w_2)}{(KC+4B)}$$

$$R = \frac{(w_1+w_2)l}{2}$$

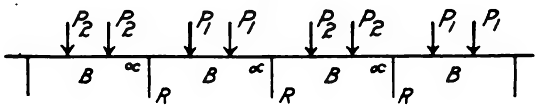
Beams of many equal spans, live loaded with a concentrated load at the center of alternate bays, and monolithic with the columns.



$$\propto = -\frac{l}{8E} \cdot \frac{(P_1 - P_2)}{(KC + 4B)}$$

$$R = \frac{P_1 + P_2}{2}$$

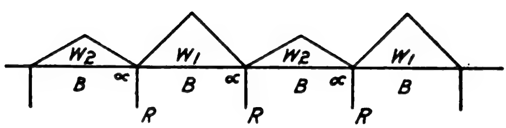
Beams of many equal spans, live loaded on alternate bays with loads at the third points for both live and dead loads, the beam being monolithic with the columns.



$$\propto = -\frac{2l}{9E} \cdot \frac{(P_1 - P_2)}{(KC + 4B)}$$

$$R = P_1 + P_2$$

Beam of many equal spans, live loaded on alternate bays with a triangular distribution of both live and dead loads, the beam being monolithic with the columns.



$$\propto = -\frac{5l}{48E} \cdot \frac{(W_1 - W_2)}{(KC + 4B)}$$

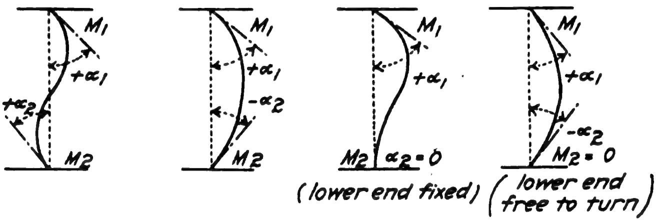
$$R = \frac{W_1 + W_2}{2}$$

Column Formulas

A column may take any of the positions shown on page 401 (deflections magnified).

The formulas given for a beam of one span, uniformly loaded,

may be employed to find the moments in a column when the end slopes are known. For columns, the lateral load w becomes zero,



and we may reduce the formulas mentioned to the following simple terms:

Case (a). When $\alpha_1 = \alpha_2$

$$M_1 = -6EB\alpha_1$$
$$M_2 = +6EB\alpha_1$$

Case (b). When $\alpha_1 = -\alpha_2$

$$M_1 = -2EB\alpha_1$$
$$M_2 = +2EB\alpha_1$$

Case (c). When $\alpha_2 = 0$

$$M_1 = -4EB\alpha_1$$
$$M_2 = +2EB\alpha_1$$

Case (d). When $M_2 = 0$

$$M_1 = -3EB\alpha_1$$

The general expression for the moment at one end may be written

$$M = KEC\alpha$$

in which K will vary from 2 to 6.

72. Unintentional Eccentricity.—Columns in ordinary building construction are subject to unsymmetrical floor loading and such loading may be termed unintentionally eccentric.

Interior Columns

Two Spans.—Consider the case where the left-hand span has a uniform load of w_t and the right-hand span a load of w_d per unit of length, where

$$w_t = \text{total load per unit length}$$
$$w_d = \text{dead load per unit length}$$

If the stiffness of the columns is neglected and the beam is considered as freely supported, we have from Art. 71

$$\alpha = \frac{(w_t - w_d)l^2}{48EB}$$

$$R = \frac{5(w_t + w_d)l}{8}$$

(The moment diagram for this case, considering w_d equal to zero, is given in Fig. 294.)

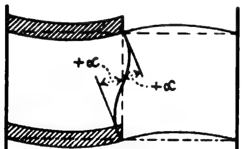
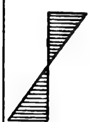
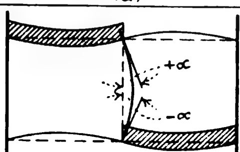
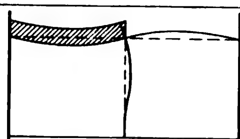
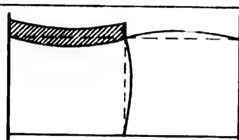
Lower End of Column	Configuration	Bending Moment Diagram	B.M. at Top of Column	B.M at Bottom of Column
Constrained to take up a slope = $+\alpha$	 (a)		$-6EC\alpha$	$+6EC\alpha$
Constrained to take up a slope = $-\alpha$	 (b)		$-2EC\alpha$	$-2EC\alpha$
Fixed	 (c)		$-4EC\alpha$	$+2EC\alpha$
Free to Turn	 (d)		$-3EC\alpha$	Zero

Fig. 315.

The column may take any of the positions shown in Fig. 315 depending upon the condition of the lower end. Consider the lower end fixed (Case c) which is the case illustrated in Fig. 316. (The arrangement in the latter figure has frequently been adopted in factories, the beam being continuous over a central row of columns and supported at the ends in brick walls. The beam should be assumed to have free ends for the reason that it is diffi-

cult to completely fill pockets in brick walls with concrete.) The column design as shown is satisfactory according to the common method of figuring—that is, assuming both spans fully loaded. It becomes important to determine the stress in the column due to the live load on one span only. After this stress is found, the exact stress for full loading will be determined and the stresses compared.

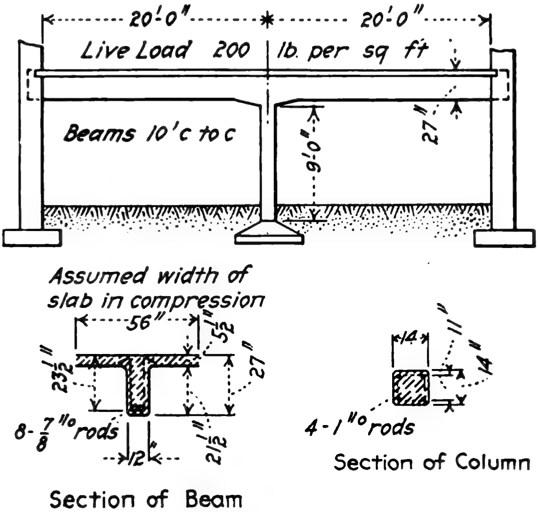


FIG. 316.

$$w_d = (69)(10) + \frac{(12)(21.5)(150)}{144} = 960$$

$$w_t = (200)(10) + 960 = 2960$$

Live load on one span:

$$R = \frac{5(2960 + 960)(20)}{8} = 49{,}000 \text{ lb.}$$

Keeping all dimensions in pounds and inches,

$$B = \frac{I_b}{l} = \frac{(0.036)(56)(23.5)^3}{(20)(12)} = 109 \qquad \text{(See Fig. 317)}$$

$$\alpha = \frac{\left(\frac{2000}{12}\right)(20)^2(12)^2}{(48)(109)E} = \frac{1835}{E}$$

We have seem that if the column is considered as fixed in

direction at its lower end, K may be taken as 4, and hence the moment induced in it by this slope will be

$$M = 4EC\alpha$$

or

$$M = 4E\left(\frac{I_c}{9\times 12}\right)\left(\frac{1835}{E}\right) = 68.0\ I_c$$

A fireproof covering of 1½ in. will be assumed on the column and no account will be taken of this in the computations, although the stress due to bending will be somewhat greater at the outside of the column than at the edge of the effective area. For convienence, the steel will also be assumed 1½ in. from the face of the column.

$$\text{The stress due to bending} = \frac{Mx_1}{I_c}^{*}$$

$$= 68.0\ I_c \times \frac{5.5}{I_c}$$

$$= 374 \text{ lb. per square inch.}$$

It should be clear from the above that it is not necessary to find the moment of inertia of the column when the column stiffness is neglected in calculating the slope of the beam.

The equivalent concrete area of the column is (assuming $n = 15$),

$$121 + (3.14)(14) = 165.0 \text{ sq. in.}$$

The stress due to direct compression $= \frac{49{,}000}{165.0} = 297$ lb. per square inch.

$\therefore$ maximum stress $= 671$ lb. per square inch.

(*Note.*—The minimum stress is tension and is somewhat above the allowable tensile stress for concrete, but there is no necessity here for using the more complex formulas which apply when the tensile strength of the concrete is neglected.)

Live load on both spans:

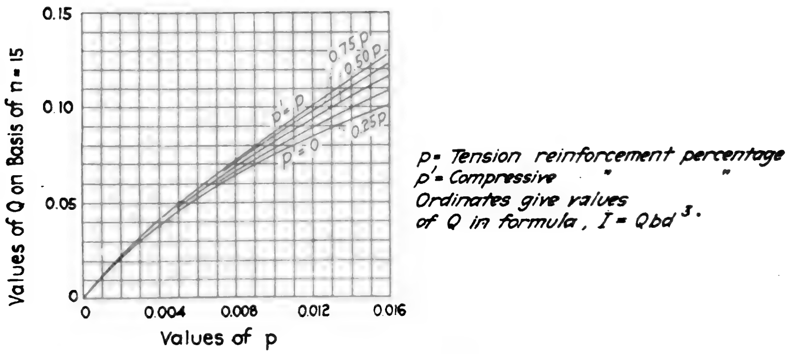
$$R = 1.25\ w_t l = 1.25\ (2960)(20) = 74{,}000 \text{ lb.}$$

stress $= \frac{74{,}000}{165.0} = 448$ lb. per square inch (designed for 1 : 2 : 4 concrete, $f_c = 450$)

* See Art. 74 of Volume I.

It is obvious that the column should be made stronger in order to bring the maximum stress in the concrete to a proper working value. The maximum stress occurs for only one span covered with live load, and this stress in considerably greater than for full loading, although the beams are stiff and the column relatively flexible.

When beams are both long and shallow, a much greater slope will be produced, and consequently greater eccentricity in the column. Greater eccentricity will also occur for columns which are relatively short and stiff.



Curves for Computing Ratio of Moments of Inertia in Rectangular and T-Beams

For different percentages of tension reinforcement and five ratios of compressive reinforcement

FIG. 317.

A case that is worse than either of those investigated occurs when the live load extends continuously over one span and partly over the other. The additional live load over only part of a span has little influence on the slope of the beam but increases considerably the direct load on the column.

When the columns are very stiff compared to the beam, it becomes necessary to take their stiffness into account in calculating the slope. For two spans rigidly connected to a central column and freely supported on walls at the ends, we have from Art. 71

$$\alpha = \frac{(w_l - w_d)l^2}{8E(KC+6B)}$$

This formula will be applied to a building having three floors and a basement, the floors and roof all being designed for the same live load. For convenience, the spans, height of columns, etc., will be taken the same as in the last example and only the two lower columns (b and c, Fig. 318) will be considered. In the problem which follows, the dead weight of the columns will be neglected.

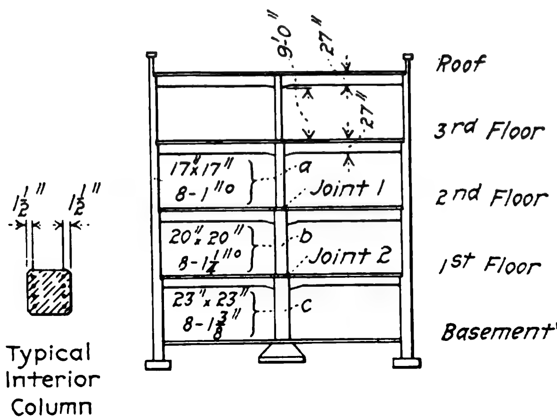


Fig. 318.

When a column extends above and below the joint with a beam, KC in the formula is to be replaced by $K_aC_a+K_bC_b$; K_a and C_a referring to the column above, and K_bC_b referring to the column below the joint at which the slope of the beam is found. This should be done since it is obvious that the restraining action of a column is increased when the column above has to be bent as well as that below.

The value of K to apply to any particular column needs careful attention. Referring to Fig. 315 it will be seen that a value of 6 is necessary if two adjacent floors are unequally loaded. For K, however, to be taken as 6 for both upper and lower columns at any given joint, three adjacent floors would need to be unequally loaded to the fullest extent. It becomes a question of judgment as to what conditions are likely to occur. It will be assumed in the building under consideration that the worst case that needs attention is for the floor above and below to be horizontal at the joint. This condition gives a value of $K=4$.

The greatest stress in a column due to bending and direct load

will occur when the floor, at which the joint is taken, is only half loaded and when all the floors above are fully loaded. This direct load on column b is

$$(2)(74{,}000)+49{,}000=197{,}000 \text{ lb.}$$

and on column c is

$$197{,}000+74{,}000=271{,}000 \text{ lb.}$$

Joint 1—Live load on one span:

$$B=109$$

$$C_a=\frac{\frac{1}{12}(14)(14)^3+12(6.28)(7)^2}{(9)(12)}=64$$

$$C_b=\frac{\frac{1}{12}(17)(17)^3+12(9.82)(8.5)^2}{(9)(12)}=143$$

$$K_aC_a+K_bC_b=4(64+143)=828$$

The moment of inertia of the columns is calculated on the assumption that $n=12$ and that the steel is placed along two opposite faces of the column.

$$\propto=\frac{\left(\frac{2000}{12}\right)(20)^2(12)^2}{8E(828+654)}=\frac{810}{E}$$

Stress in column b due to bending $=\frac{Mx_1}{I}=\frac{4E\propto x_1}{l}$

$$=\frac{(4)(810)(8.5)}{(9)(12)}$$

$=255$ lb. per square inch.

The equivalent concrete area of the column is

$$(17)(17)+(9.82)(11)=397.0 \text{ sq. in.}$$

The stress due to direct compression $=\frac{197{,}000}{397.0}=496$ lb. per square inch.

$\therefore$ maximum stress $=751$ lb. per square inch.

Joint 1—Live load on both spans:

$$R=(74{,}000)(3)=222{,}000 \text{ lb.}$$

$\text{stress} = \dfrac{222{,}000}{397.0} = 560$ lb. per square inch (designed for $1:1\frac{1}{2}:3$ concrete, $f_c = 560$)

Joint 2—Live load on one span:

$$C_c = \frac{\frac{1}{12}(20)(20)^3 + 12(11.88)(10)^2}{(9)(12)} = 255$$

$$K_bC_b + K_cC_c = 4(143 + 255) = 1592$$

$$\alpha = \frac{\left(\frac{2000}{12}\right)(20)^2(12)^2}{8E(1592+654)} = \frac{534}{E}$$

$$\text{Stress in column } c \text{ due to bending} = \frac{4E\alpha x_1}{l}$$

$$= \frac{(4)(534)(10)}{(9)(12)}$$

$$= 198 \text{ lb. per square inch.}$$

The equivalent concrete area of the column is

$$(20)(20) + (11.88)(11) = 530.7 \text{ sq. in.}$$

The stress due to direct compression $= \dfrac{271{,}000}{530.7} = 510$ lb. per square inch.

$\therefore$ maximum stress $= 708$ lb. per square inch.

Joint 2—Live load on both spans:

$$R = (74{,}000)(4) = 296{,}000 \text{ lb.}$$

$\text{stress} = \dfrac{296{,}000}{530.7} = 557$ lb. per square inch (designed for $1:1\frac{1}{2}:3$ concrete, $f_c = 560$)

If the ends of the beams were supported by concrete wall columns, the restraint at the beam ends would reduce the slope at the interior columns. (This would, in fact, be true for beams of any number of spans.) Thus, the use of formulas for ends freely supported will always be on the side of safety. If it is desired to take the reduction into account, the correct expression for the slope may be taken from Art. 71 for the usual case where the outside columns are similar.

Three Spans.—Formulas are given in the preceding article for the slope at the interior columns in continuous beams of three spans, ends freely supported. These apply to the usual case where the beams and columns are symmetrical about the center

line of the center beam. A general expression for three-span beams is very complex and hence the reason for the less general expressions given. As in the case of two spans, the use of formulas for freely supported ends are on the safe side.

The student should also notice that the formulas given apply when the two end spans are fully loaded and the middle span unloaded except for the dead load. This loading has been found to give the worst condition of bending on the columns. The formulas include the case where the end spans are designed for $\frac{wl^2}{10}$ and the center span for $\frac{wl^2}{12}$. The method of finding the stress in the columns is similar to that explained for two-span beams.

Four or More Spans.—In the case of beams of four or more spans it is sufficiently accurate to treat the first interior columns in the same way as the corresponding columns under beams of three spans. For the other interior columns, the influence of the end spans on the eccentricity in such columns is very small, and the formulas for the slope in a beam of an infinite number of spans may be used.

Exterior Columns

One Span.—When a building contains several floors, simultaneous loading must be provided for. The value of $K=6$, for columns above the first floor, assumes that the slope at the end of different floors is equal. This is not accurate, as the slope generally diminishes in the lower floors, owing to the increased stiffness of the columns. Generally no great error is made by taking this value for K but, if desired, a process of trial and error may be employed to determine the exact value of K for any particular case. The reaction is, of course,

$$R=\frac{wl}{2}$$

and the stresses are calculated as in the previous examples.

By using the formulas of Art. 71, it has been found that the size of outside columns should be determined much more by considerations of bending than by those of direct load. Also, it has been fouud that it is good design *not* to vary the outside columns greatly through several stories since, even for uniform

column section throughout, the stress in the upper portions of the column is approximately the same for ordinary conditions as in the lower portions. A smaller load usually comes upon the upper floors but this is offset by the smaller amount of restraint afforded to the shallower beams.

Two Spans.—The value of $\propto_1$ is usually about 25 per cent less for two spans than for one span. The greatest eccentricity on column 1 occurs when the left-hand span is fully loaded and the right-hand span has its dead load only—the same loading that gives maximum bending in the interior column.

Three or More Spans.—In the case of three spans, the greatest eccentricity on the outside columns occurs when the end spans are fully loaded and the central span is unloaded. This condition is the same as that which gives the greatest bending in the interior columns.

In the case of four spans, the worst case for column 1 occurs when the live load extends over spans 1 and 3, and only the dead load over spans 2 and 4. The actual effect of the fourth span is to reduce the eccentricity of column 1 but the difference is extremely small. The formulas given in Art. 71 for the case of three spans may be used for four or more spans without appreciable error.

Problem

23. Consider the building shown in Fig. 318 to have three spans with the loading, spans, height of columns, sizes of beams, etc., the same as in the illustrative example previously given. (a) Find the stress in the interior columns *b* and *c* considering the ends of the beams as freely supported and $B_1 = B_2$. (b) Assume interior columns of concrete with $K_2C_2 = K_1C_1$. Find the stress in outside lower column.

73. Intentional Eccentricity.—Columns are frequently subjected to bending moments greatly in excess of any arising from slight unintentional eccentricity in the application of the load. For example, a column supporting a roof is frequently made to carry a traveling crane which runs on a track supported by side brackets or at one side of the column (Figs. 151A and 151B). To compute the maximum stress in such a column, it is necessary to find the maximum bending moment at whatever section it occurs, and then combine the stresses due to bending and thrust by the general method explained in Chapter IX of Volume I.

The maximum bending moment occurs *at* the load, and depends upon the height at which the load is placed and the end conditions of the column. To simplify the calculations, the depth of the bracket will be considered small in comparison with the length of the column. This is not strictly correct but the error involved will be on the safe side since any increase in the depth of a bracket reduces the maximum amount.

The bending stress due to loads intentionally eccentric usually cause tension to exist over part of the section which is greater than the allowable tensile strength of the concrete. When Diagrams 14 and 15 of Volume I cannot be employed, the quickest way to obtain the resulting stress (neglecting the concrete in tension) is to follow the method outlined in Art. 76 of Volume I and determine the position of the neutral axis by trial.

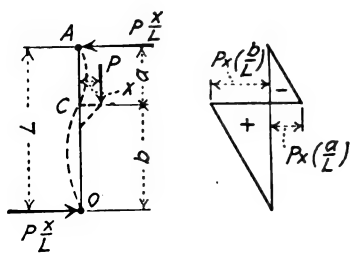


FIG. 319.

Columns supporting bracket loads will be considered for the three conditions: (1) both ends free to turn, (2) both ends fixed, and (3) one end fixed and the other end free. Columns in practice will have conditions intermediate to these and good judgment as to flexibility of the end connections is necessary to arrive at correct results in any particular case.

Column Free to Turn at Both Ends.—The bending-moment diagram for this case is shown in Fig. 319. The bending moment, Px, of the eccentric load is resisted by horizontal forces at the ends of the column which form a couple, the value of which is also Px. The maximum possible value is evidently Px, which would occur with the load at either end of the column. The minimum bending moment occurs when $a=b=\frac{L}{2}$, and equals $\frac{1}{2}Px$.

Column Fixed at Both Ends.—Analysis shows that the least value of the bending moment is $\frac{1}{2}Px$ and occurs when b has the following values: $0211L$, $0.500L$, and $0.789L$. The maximum moment is Px and occurs when the bracket is either at the top or bottom of the column. The moment diagrams for these cases are shown in Fig. 320.

The following general formulas may also be obtained:

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At A (Fig. 320),

$$M_A = Px\frac{b}{L}\left(2-3\frac{b}{L}\right)$$

$$R = \frac{6Pxb}{L^2}\left(1-\frac{b}{L}\right)$$

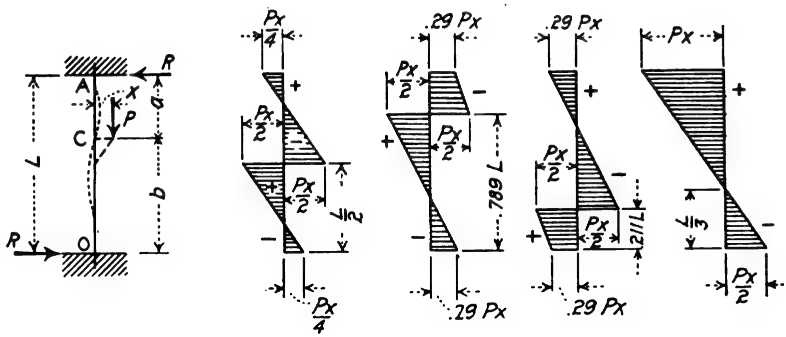


FIG. 320.

Just above C,

$$M = M_A - Ra$$

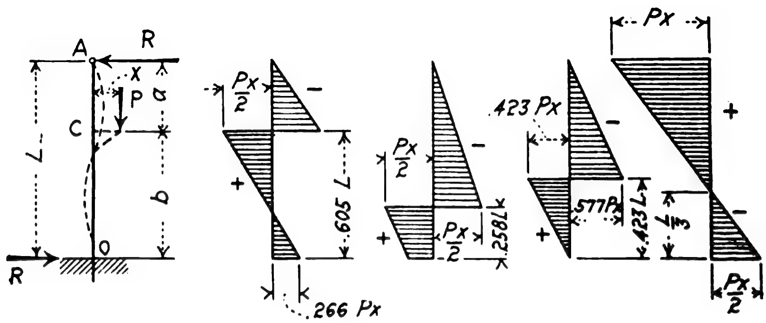


FIG. 321.

Just below C,

$$M = M_O + Rb$$

At O,

$$M_o = Px\left[1-4\left(\frac{b}{L}\right)+3\left(\frac{b}{L}\right)^2\right]$$

Column Fixed at One End and Free to Turn at the Other.—The minimum bending moment of $\frac{1}{2}Px$ occurs for b equal to $0.258L$ or $0.605L$. The bending-moment diagrams for these cases are shown in Fig. 321 which also shows the condition to make the bending moment at the base zero, and the particular case of $b=L$.

The following general formulas apply:

$$R=\frac{3Pxb}{2L^2}\left(2-\frac{b}{L}\right)$$

Just above C (Fig. 321),

$$M=-Ra$$

Just below C,

$$M=Px-Ra$$

PROBLEM

24. Design a rectangle column 15 ft. long to have fixed ends and to support a 10,000-lb. load on a bracket placed at two-thirds the height of the column from the base. The load must be placed 4 in. out from the edge of the column.

CHAPTER XV

WIND STRESSES

74. General Discussion.—In high and narrow buildings attention should be given to the stresses caused by wind. This matter is often neglected in reinforced-concrete buildings, but the bending and direct stresses in columns and girders due to wind load are frequently considerable and should be provided for.

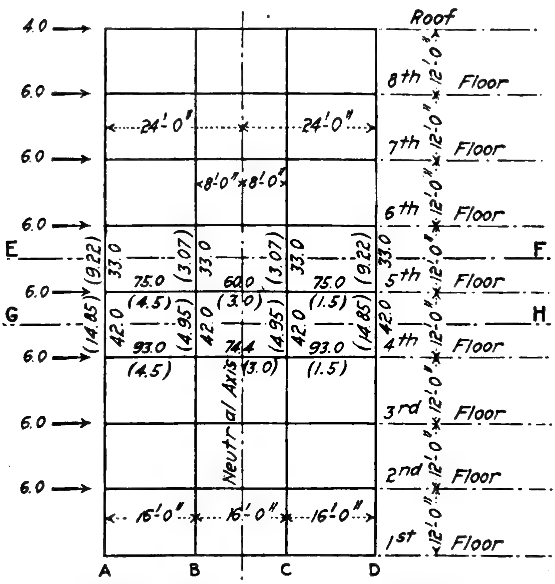
Unfortunately a rigid solution for finding wind stresses, applying the exact theory of framework with rectangular panels, involves equations entirely too long and cumbersome for use in practice. There are, however, a number of approximate methods in current use in the design of tall steel buildings which may be employed for concrete buildings to give safe results. All such methods, of course, are open to the criticism that the assumptions made are not all in accordance with the facts. The method to be employed in this chapter seems to the writer more rational than any of the others.

75. An Approximate Method.—A single bent will be considered and for the present, it will be assumed that all columns in any given story are identical, that all girders of the same floor have exactly the same design with a constant moment of inertia throughout, and that the joints are perfectly rigid. It will be also assumed that the longitudinal deformations of the members may be neglected and that the points of inflection of the columns occur at the mid-points.

A high narrow building subjected to wind pressure is similar to a cantilever beam loaded at frequent intervals. All columns on the windward side of the building will take a direct stress of tension and on the leeward side a direct stress of compression. With equal column spacings, what might be called a neutral axis will occur at the center of the building (Fig. 322), and the direct stress in any given column will be proportional to the distance of the column from this neutral axis. The shear on any horizontal plane will be assumed equally distributed among the columns cut by that plane.

A section or bent of a high narrow building is represented in Fig. 322. To be able to find the internal stresses in the bent due to wind loads, it is necessary to find the total horizontal shear and the bending moment of the building at any given line of inflection of the columns. The moment of the wind loads must be placed equal to the moment of the direct stresses in the columns about the neutral axis.

The calculation of stresses and bending moments in members between the fourth and sixth floors will be given in detail. Let



Loads and stresses are given in thousands of pounds and bending moments in thousands of foot-pounds. Direct stresses are given in parenthesis ( ).

FIG. 322.

$8X$ be the direct stress in each of the fifth-story columns B and C, then $24X$ will be the direct stress in each of the fifth-story columns A and D. Taking moments about the line of inflection, EF, we have

$$(4000)(42)+(6000)(30+18+6)=(24X)(24+24)+(8X)(8+8)$$

From which $8X=3075$ lb., and $24X=9225$ lb.

In the same way for the fourth-story columns we have, taking moments about the line of inflection GH,

$$(4000)(54)+(6000)(42+30+18+6) = (24X)(24+24)+(8X)(8+8)$$

From which $8X = 4950$ lb., and $24X = 14{,}850$ lb.

The total horizontal shear on any line across the building is the sum of the wind loads above that line. The total shear in the fifth-story columns, for example, is 22,000 lb. and in the fourth-story columns is 28,000 lb. (Fig. 323). The total horizontal shear in any given story will be assumed equally distributed among the columns.

Direct stresses and shears in the columns of the fourth and fifth stories are given in Fig. 323. The shear in the girders AB and CD of the fifth floor is $14{,}850 - 9225 = 5625$ lb. The shear in the girder BC is $(14{,}850-9225)+(4950-3075) = 7500$ lb.

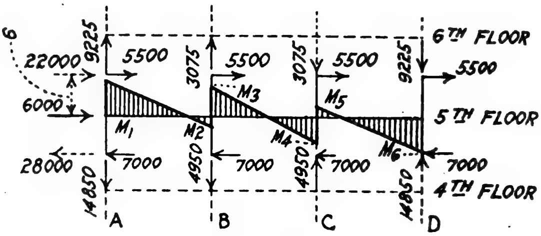


FIG. 323.

The equations for bending moments in the girders can be written as follows:

$$M_1 = M_6 = (5500)(6)+(7000)(6) = +75{,}000 \text{ ft.-lb.}$$
$$M_2 = M_5 = +75{,}000 - (14{,}850\text{-}9225)(16) = -15{,}000 \text{ ft.-lb.}$$
$$M_3 = M_4 = -15{,}000 + 75{,}000 = +60{,}000 \text{ ft.-lb.}$$

The bending moment at the fifth-floor girder of each fifth-story column is $5500 \times 6 = 33{,}000$ ft.-lb., and of each fourth-story column (assuming the girder to have no depth) is $7000 \times 6 = 42{,}000$ ft.-lb.

The compression in the floor girders is $6000 - 1500 = 4500$ lb. between columns A and B, $4500 - 1500 = 3000$ lb. between B and C, and $3000 - 1500 = 1500$ lb. between C and D.

If a high building is not long in proportion to its width, the building should be calculated for wind in a longitudinal direction as well as transversely. The reason for this is obvious.

If the columns are unequally spaced or their equivalent concrete areas are different, the location of the neutral axis must first

be found. The direct stresses in the columns will vary both as their distances from the neutral axis and their transformed sectional areas. The horizontal shears taken by the columns will vary as their moments of inertia. A proof of this statement regarding the horizontal shears may be found in an article by Prof. Clyde T. Morris in the issue of *Engineering News* of April 24, 1913.

Consider the spacing of columns as shown in Fig. 324 and assume the transformed sectional areas of the columns to be as follows:

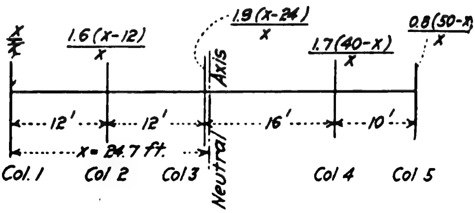


FIG. 324.

Col. 1	A sq. in.
Col. 2	1.6A sq. in.
Col. 3	1.9A sq. in.
Col. 4	1.7A sq in.
Col. 5	0.8A sq. in.

The unit stress on each column will be proportional to its distance from the neutral axis, or distance from the neutral axis divided by x (Fig. 324). On each side of the neutral axis the sum of the loads acts as a resultant at a certain distance from the neutral axis. These resultants are equal and act in opposite directions. Since the two resultants are equal:

$$\frac{x}{x}+\frac{1.6(x-12)}{x}+\frac{1.9(x-24)}{x}=\frac{1.7(40-x)}{x}+\frac{0.8(50-x)}{x}$$

$$x=24.7 \text{ ft.}$$

With x known, the stresses may be determined in the same manner as in the case above, where the columns are equally spaced. Let the total stress in Col. 1 be denoted by X and the moment at the line of inflection in question be denoted by M, then

$$M=24.7X+\frac{1.6(12.7)(X)}{24.7}(12.7)+\frac{(1.9)(0.7)(X)}{24.7}(0.7)+$$
$$\frac{1.7(15.3)(X)}{24.7}(15.3)+\frac{(0.8)(25.3)(X)}{24.7}(25.3)$$

The stress X in Col. 1 may be found from this equation. Then the stress in Col. 2 equals $\frac{(1.6)\,(12.7)}{24.7}(X)$, and so on.

76. Wind Loads and Working Stresses.—As in the design of steel buildings it is the custom to allow working stresses 50 per cent greater for wind stresses than are allowed for the customary live and dead loads, and to assume a horizontal wind pressure of 30 lb. per square foot. This is equivalent to using the same working stresses throughout and providing for a wind pressure of 20 lb. per square foot. Some engineers neglect the wind stresses when the combined stresses due to live-, dead-, and wind-loads do not increase by 50 per cent the stresses due to live- and dead-loads alone. The wind-loads and working stresses, however, are seldom left to the choice of the designer, being generally fixed by the building code of the city in which the building is located.

77. Design of Individual Members.—Columns should be designed (in connection with the live- and dead-loads) to take the direct stress due to wind pressure, and this direct stress on the tension side of the neutral axis should not be permitted to exceed the compressive stress due to dead load. The bending moment of any column at the floor girder is found, as already explained, by multiplying the total amount of shear acting at the point of inflection of the column by its distance to the floor. Extra rods are usually provided to care for the bending moments in columns and these rods are lapped at the points of inflection—that is, at the mid-points of the columns. The column shears need to be considered in but few cases.

The compression in the floor girder is due entirely to the wind-load at the floor in question. Each column takes off a portion of this load, thus reducing the compression in each span from windward to leeward. This compression in the floor girders is usually so small as to be negligible. The shears and bending moments in floor girders are, however, often quite important. In order to design accurately for moment it is usually necessary to plot moment diagrams for both wind-load and floor-load and then the total moment at any point may be readily determined by scaling.

PROBLEM

25. Compute the moments and direct stresses in all members shown in Fig. 322.

CHAPTER XVI

DESIGN OF A FACTORY BUILDING

Plates XVI to XXIII inclusive on the design of a factory building are not much more than suggestive and are presented in order to give the student the proper start in working out the problems at the end of this chapter. The typical interior floor-bay design for this building is given in detail in Art. 17, and the typical roof-bay design in Art. 38. The typical interior column and the typical interior and wall column footings are presented in Arts. 43 and 47 respectively. Plate XXIII is the proposal freight-elevator layout, drawn especially for this book at the Chicago office of the Otis Elevator Co.

The window dimensions have been taken from tables furnished by one of the prominent manufacturers of steel sash. Steel sash are made in standard units of three, four, and five lights wide and in any desired number of lights in height up to 15 ft. By using these standard units and combining them by means of mullions, openings of almost any desired width and height can be fitted. The tables above-mentioned are given below and the student should notice that all window openings in the building have been designed with the object in view of using this particular type of steel sash. Units designated as *A* are three lights wide; *B*, four lights wide; and *C*, five lights wide. The windows between typical wall columns consist of four *C* units five lights high, each pane of glass 10 in. by 17 in. The masonry dimensions of the windows are slightly larger that the sizes of steel sash as given in the tables, since ample clearance must be allowed at jambs to take up all inaccuracies of construction. In addition to this allowance, at least ½ in. must be allowed where ventilation extends to the jambs so that the ventilators will operate freely.

The sizes of the I-beams for the freight elevator have been computed from the proposal layout, the position of the loads being determined by scaling. Fig. 325 shows the loads which come from both upper and lower sets of I-beams at the corners of the elevator shaft. The student should use these loads in the problems which follow.

WIDTHS OF COMBINATIONS

Combination of units	Width of glass					
	10″	11″	12″	13″	14″	15″
AA	5′ 4 1/16″	5′ 10 1/16″	6′ 4 1/16″	6′ 10 1/16″	7′ 4 1/16″	7′ 10 1/16″
BB	7′ 1 1/16″	7′ 9 1/16″	8′ 5 1/16″	9′ 1 1/16″	9′ 9 1/16″	10′ 5 1/16″
CC	8′ 10 1/16″	9′ 8 1/16″	10′ 6 1/16″	11′ 4 1/16″	12′ 2 1/16″	13′ [illegible]
AAA	8′ 0¾″	8′ 9¾″	9′ 6¾″	10′ 3¾″	11′ 0¾″	11′ [illegible]
ABA	8′ 11¼″	9′ 9¼″	10′ 7¼″	11′ 5¼″	12′ 3¼″	13′ 1¼″
ACA	9′ 9¾″	10′ 8¾″	11′ 7¾″	12′ 6¾″	13′ 5¾″	14′ 4¾″
BBB	10′ 8¼″	11′ 8¼″	12′ 8¼″	13′ 8¼″	14′ 8¼″	15′ 8¼″
BCB	11′ 6¾″	12′ 7¾″	13′ 8¾″	14′ 9¾″	15′ 10¾″	16′ 11¾″
CBC	12′ 5¼″	13′ 7¼″	14′ 9¼″	15′ 11¼″	17′ 1¼″	18′ 3¼″
CCC	13′ 3¾″	14′ 6¾″	15′ 9¾″	17′ 0¾″	18′ 3¾″	19′ 6¾″
BBBB	14′ 3 7/16″	15′ 7 7/16″	16′ 11 7/16″	18′ 3 7/16″	19′ 7 7/16″	20′ 11 7/16″
BCCB	16′ 0 7/16″	17′ 6 7/16″	19′ 0 7/16″	20′ 6 7/16″	22′ 0 7/16″	23′ 6 7/16″
CCCC	17′ 9 7/16″	19′ 5 7/16″	21′ 1 7/16″	22′ 9 7/16″	24′ 5 7/16″	26′ 1 7/16″

TABLE OF HEIGHTS

No. of lights high	Height of glass																		No. of lights high
	16″		17″		18″		19″		20″		21″		22″		23″		24″		
	ft.	in.	ft.	in.	ft;	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	
1	1	4⅞	1	5⅞	1	6⅞	1	7⅞	1	8⅞	1	9⅞	1	10⅞	1	11⅞	2	0⅞	1
2	2	9⅜	2	11⅜	3	1⅜	3	3⅜	3	5⅜	3	7⅜	3	9⅜	3	11⅜	4	1⅜	2
3	4	1⅞	4	4⅞	4	7⅞	4	10⅞	5	1⅞	5	4⅞	5	7⅞	5	10⅞	6	1⅞	3
4	5	6⅜	5	10⅜	6	2⅜	6	6⅜	6	10⅜	7	2⅜	7	6⅜	7	10⅜	8	2⅜	4
5	6	10⅞	7	3⅞	7	8⅞	8	1⅞	8	6⅞	8	11⅞	9	4⅞	9	9⅞	10	2⅞	5
6	8	3⅜	8	9⅜	9	3⅜	9	9⅜	10	3⅜	10	9⅜	11	3⅜	11	9⅜	12	3⅜	6
7	9	7⅞	10	2⅞	10	9⅞	11	4⅞	11	11⅞	12	6⅞	13	1⅞	13	8⅞	14	3⅞	7
8	11	0⅜	11	8⅜	12	4⅜	13	0⅜	13	8⅜	14	4⅜	15	0⅜	15	8⅜	...	...	8
9	12	4⅞	13	1⅞	13	10⅞	14	7⅞	15	4⅞	...	...	...	...	...	...	...	...	9
10	13	9⅜	14	7⅜	15	5⅜	...	...	...	...	...	...	...	...	...	...	...	...	10

Steel rolling doors may be placed on the face of a wall or between the jambs. If placed on the face of wall, the curtain will need a width from 8 to 12 in. more than the door opening. The coil box is usually about 15 in. high and 13½ in. wide. Curtains more than 10 ft. in width and 12 ft. in height should be operated by means of an endless chain, crank, and bevel gear.

The typical roof bay was designed in Art. 38 as carrying 2 ft. of cinders. This is the maximum fill as shown on Plate XX and occurs over only a small portion of the roof. A less fill than this should properly be considered over the typical interior roof bays at least, but, for simplicity, the student may consider a 2-ft. fill

to exist over the entire roof surface. The roof slab over the freight elevator should be figured as simply supported and reinforced in two directions.

The span of the wall beams and wall girders should be taken as the net span between supports plus the total depth of the member which is being designed. All end spans of continuous beams and girders should be figured for a moment of $\frac{wl^2}{10}$ both at the center of span and at the interior support. The beam depths given by the student in his designs should not include the 1-in. floor finish. The weight of windows may be taken at 7 lb. per square foot of window area. The live load on the loading platform should be

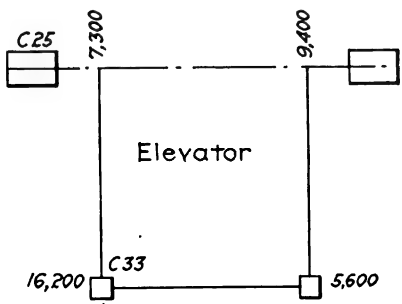


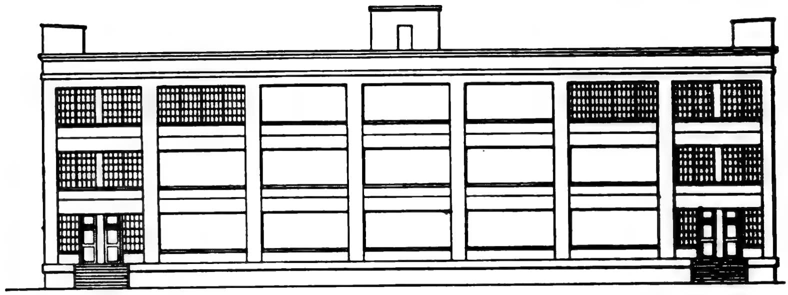
FIG. 325.

considered the same as on the floors—namely, 250 lb. per square foot including the cement finish.

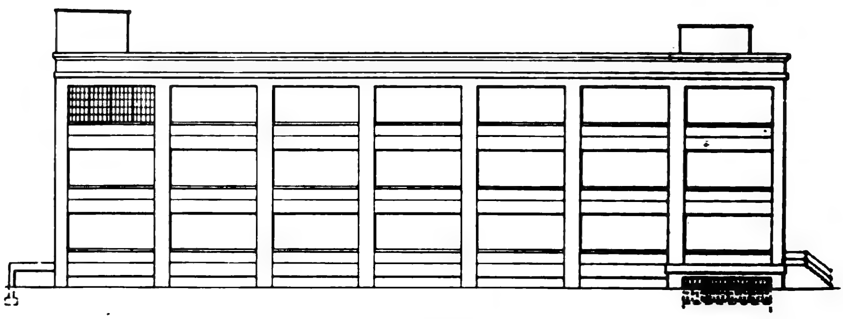
Roof girder *g* 6 at the rear of the freight elevator should be made a steel I-beam (surrounded by concrete) so as to provide properly for all diagonal-tensile stresses. It does not seem feasible to try to place reinforcing steel to take care of all such stresses in a girder so irregularly loaded. Beams *B* 11 and *B* 13, and girders *G* 6 are likely to have negative bending moment throughout their entire length and negative reinforcement should be provided throughout to take care of the resulting stresses. Beams and girders such as *B* 8, *B* 9, and *G* 7 should be figured as simply supported and six-tenths as much steel should be placed over supports as at the center of span. The ends of bent-up rods should project into adjoining slabs far enough to secure good bond.

Beam and girder data called for in the problems at the end of this chapter should be presented in the form shown on page 429:

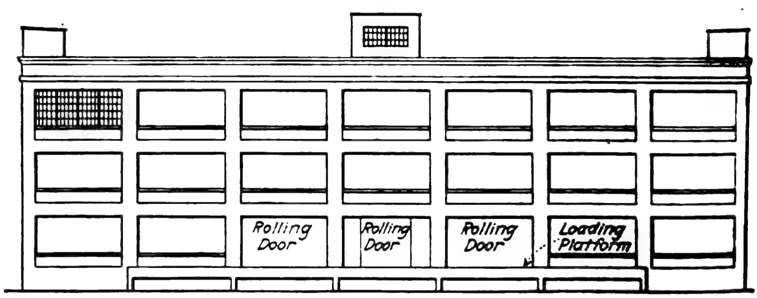
PLATE XVI



Front Elevation



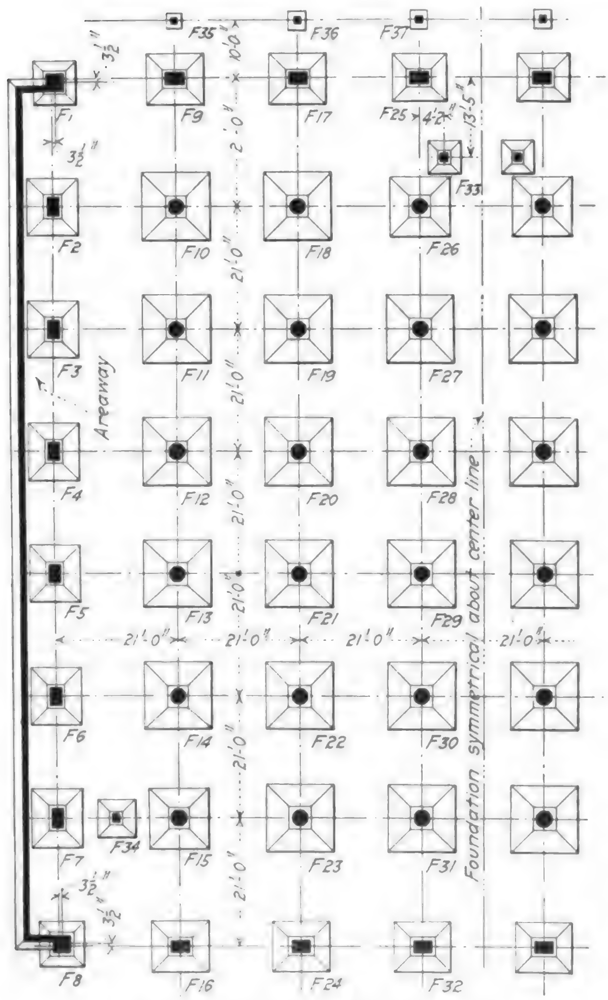
Side Elevation



Rear Elevation

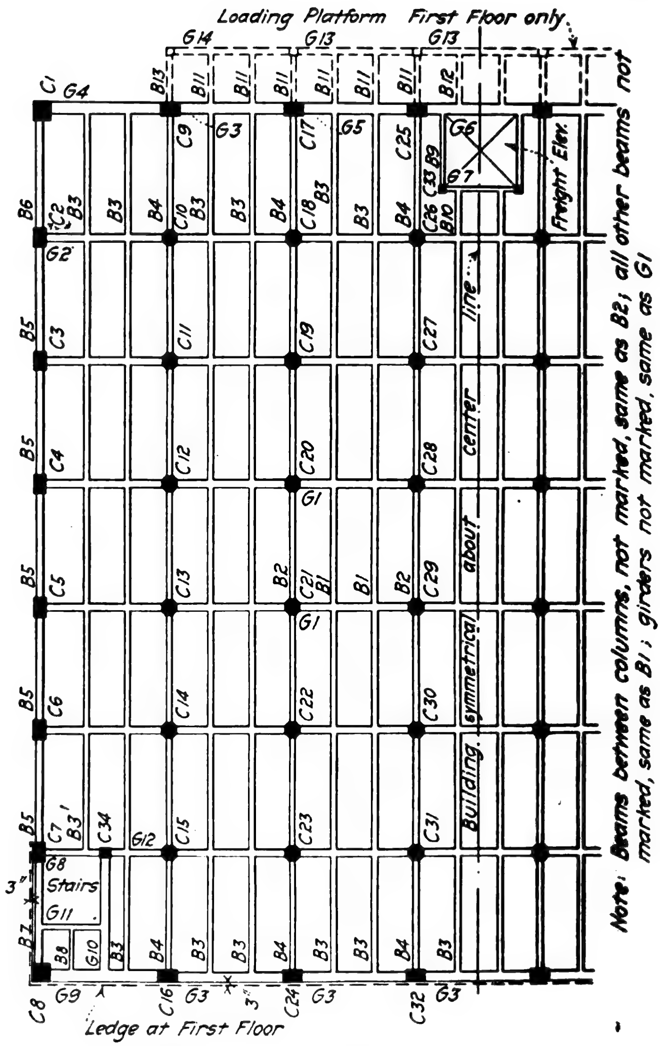
Factory building—Elevations.

PLATE XVII



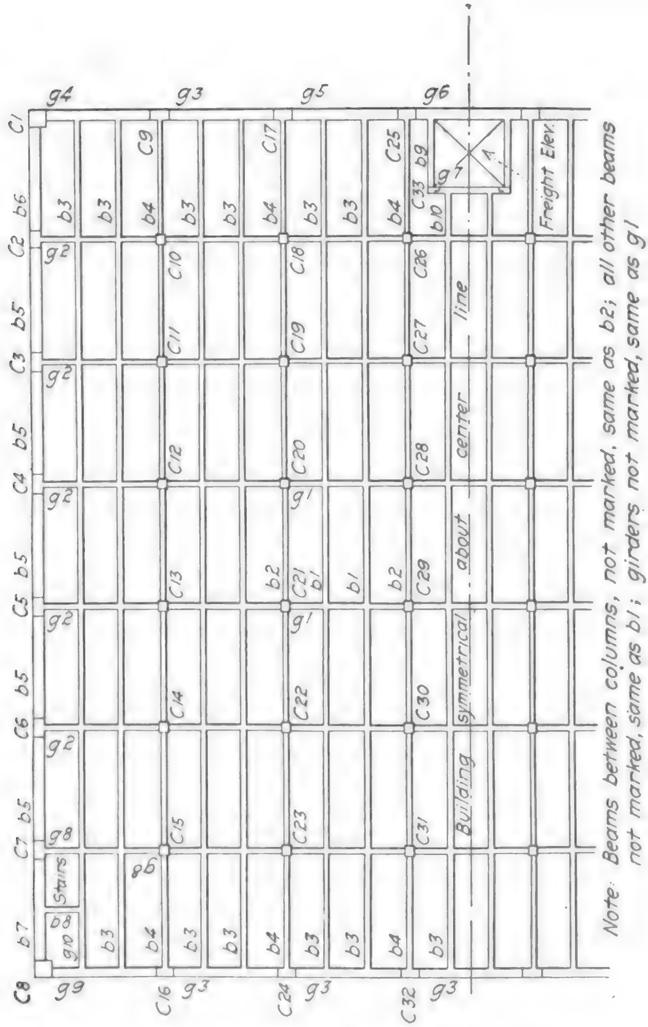
Factory building—Foundation plan.

PLATE XVIII



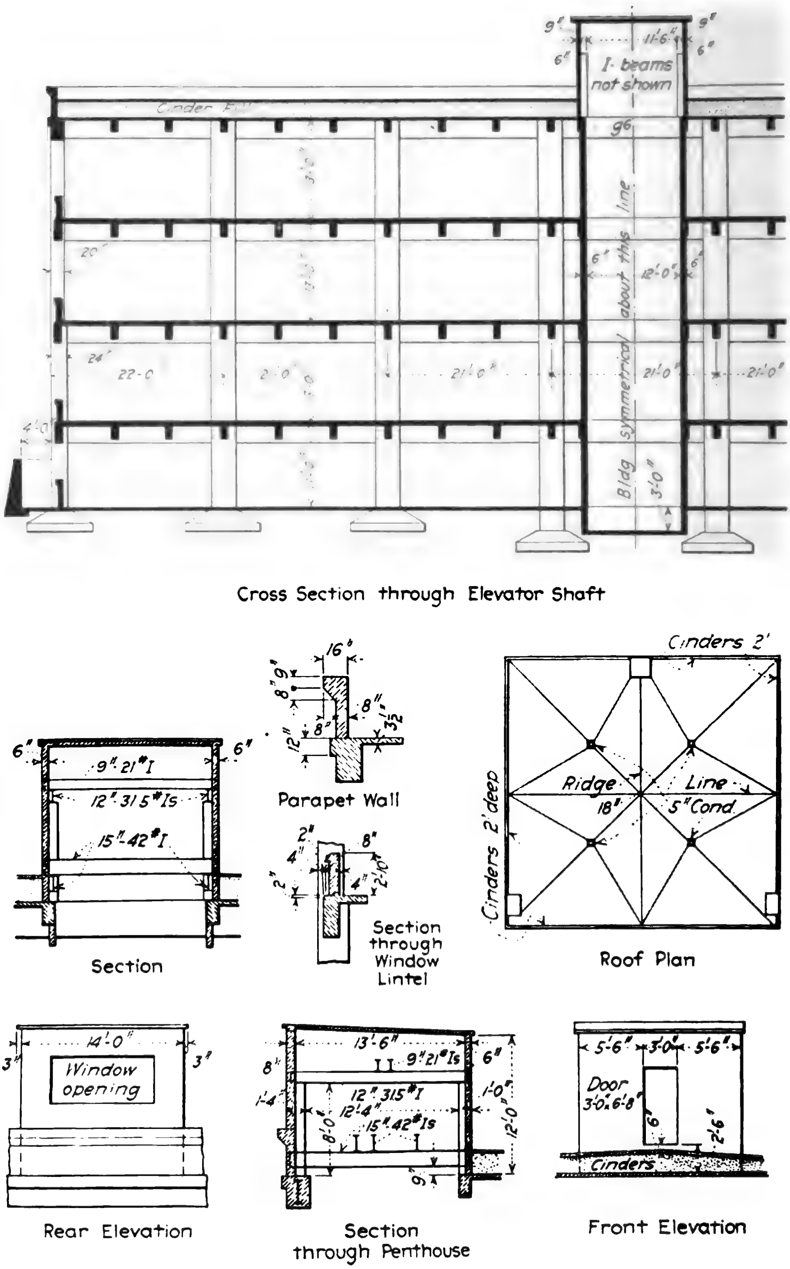
Factory building—Floor framing plan.

PLATE XIX

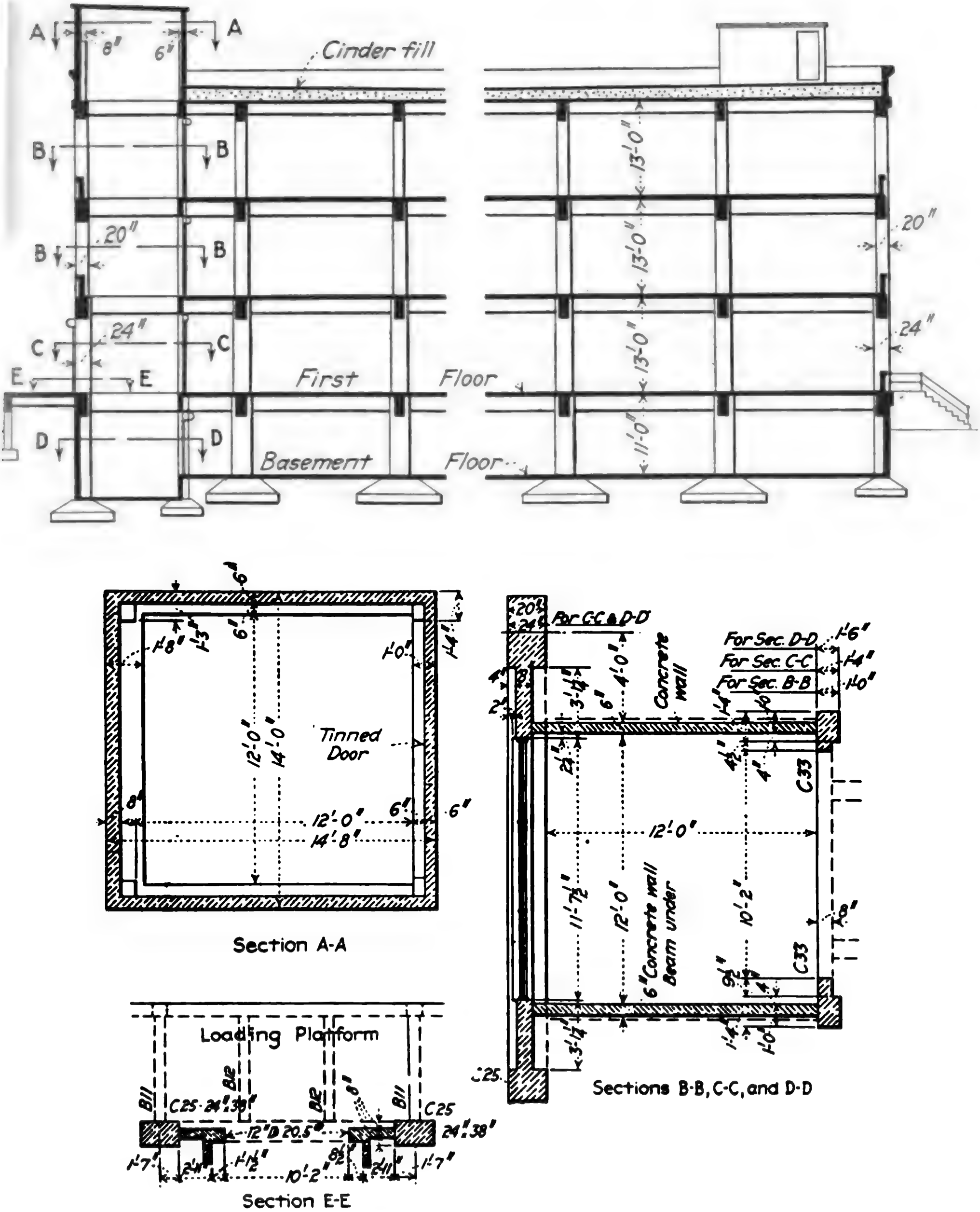


Factory building—Roof framing plan.

PLATE XX



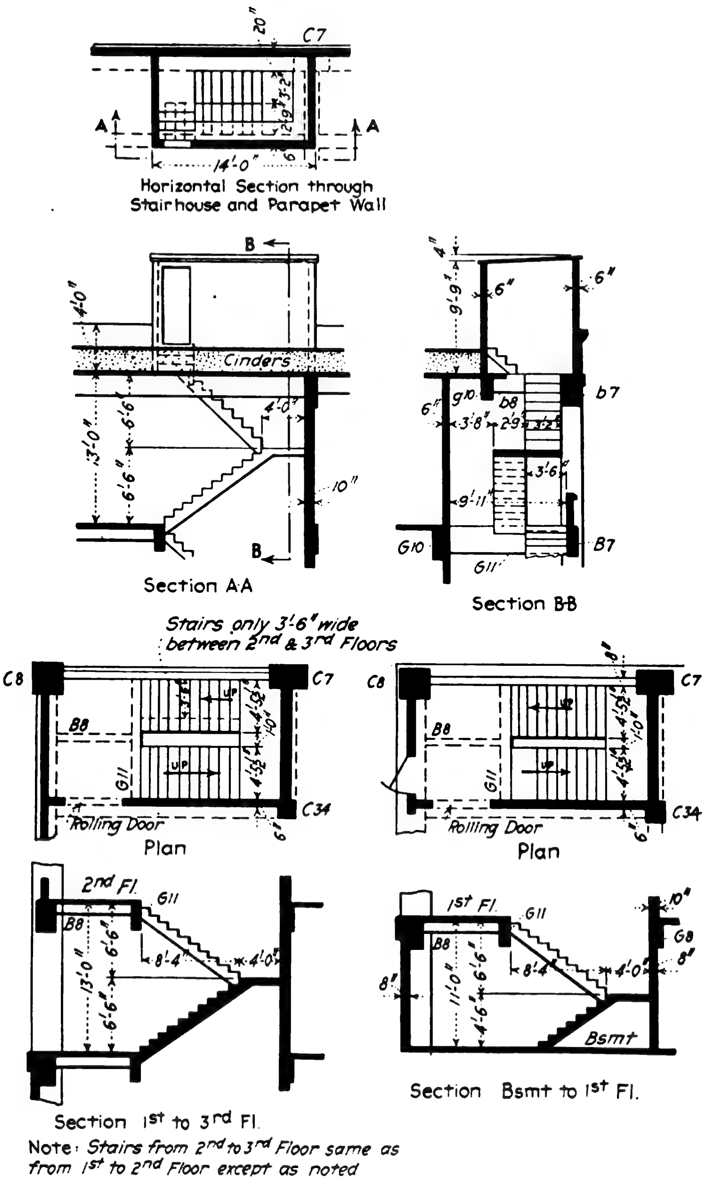
Factory building—Elevator-shaft details and roof plan.



Factory building—Cross-section and elevator-shaft details.

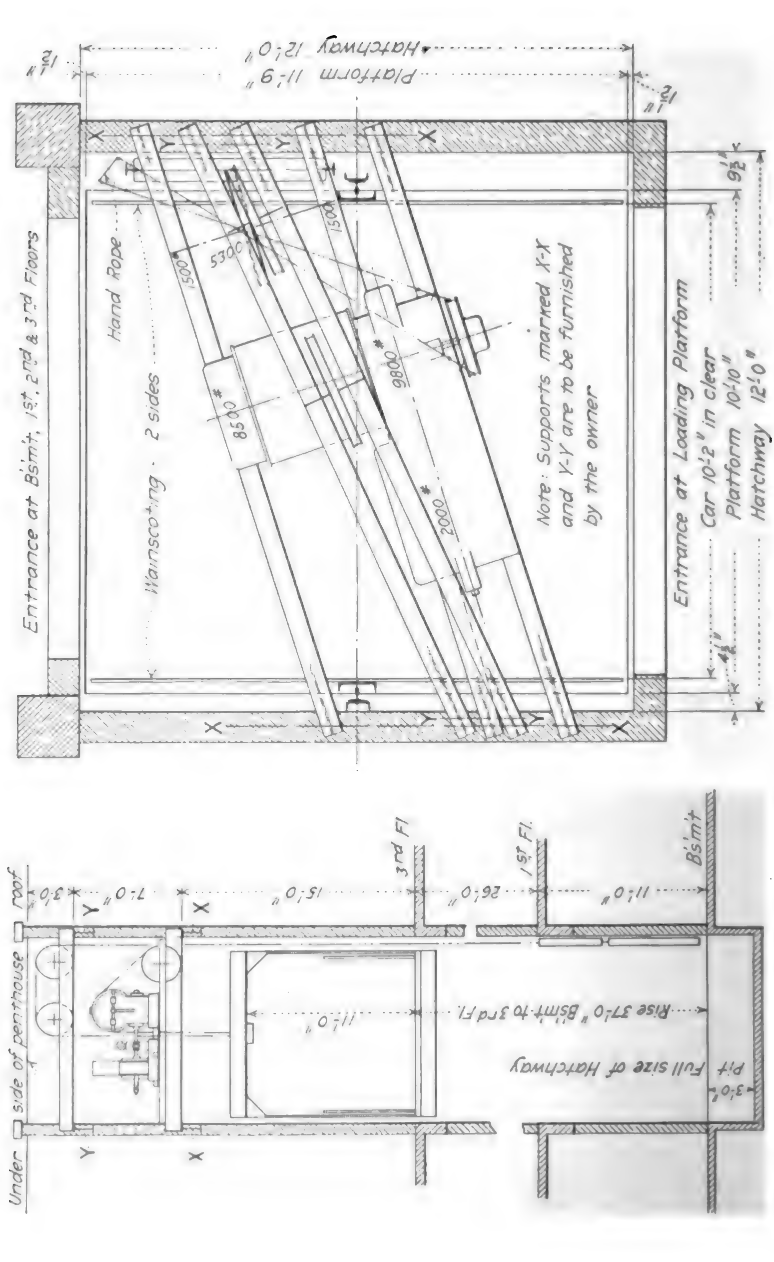
(*Facing Page* 426.)

PLATE XXII



Factory building—Stair details.

PLATE XXIII



Factory building—Proposal layout of freight elevator. Maximum load, 6000 lb. Speed, 6000 b. at 60 ft. per minute.

The data given is for the floor-bay design of Art. 17, Scheme No. 1, and this is the scheme that should be adopted by the student in the designing work which follows. Column data should be

Floors — Beam and Girder Data

Mark	Size		Depth to Steel		Reinforcement						
						Over supports					
	W	D	Center	Supports	At center	Bent up	a	b	Extra Straight Rods	Continued straight at bottom	Notes
B1 B2	10"	27"	23$\frac{1}{2}$"	23$\frac{1}{2}$"	4-$\frac{5}{8}$"° upper row 4-$\frac{3}{4}$"° lower row	2-$\frac{5}{8}$"° 2-$\frac{3}{4}$"°	2$\frac{1}{2}$"	2"		4	
B3 B3' B4	10"	27"	23$\frac{1}{2}$"	23$\frac{1}{2}$"	4-$\frac{3}{4}$"° (2 in each row) 4-$\frac{7}{8}$"° (2 " " ")	4-$\frac{3}{4}$"° outer support 4-$\frac{7}{8}$"° inner	2$\frac{1}{2}$"	2"		4	
B5	12"	32"	29$\frac{1}{2}$"	29$\frac{1}{2}$"	4-$\frac{3}{4}$"°	2	2$\frac{1}{2}$"				
G1	15"	36"	32$\frac{1}{2}$"	33$\frac{1}{2}$"	8-1"° two rows	4	1$\frac{1}{2}$"	2"		2	
G2	15"	36"	32$\frac{1}{2}$"	33$\frac{1}{2}$"	4-1"° (2 in each row) 4-1$\frac{1}{4}$"° (2 in each row)	4-1"° outer support 4-1$\frac{1}{4}$"° inner	1$\frac{1}{2}$"	2"		4 at inner support 2 at outer support	

Roof — Beam and Girder Data

Mark	Size		Depth to Steel		Reinforcement						
						Over supports					
	W	D	Center	Supports	At Center	Bent up	a	b	Extra Straight Rods	Continued straight at bottom	Notes
b1 b2	10"	21"	19"	19$\frac{1}{2}$"	4-$\frac{7}{8}$"° Rods	2	1$\frac{1}{2}$"			2	
b3 b4	12"	21"	19"	19$\frac{1}{2}$"	4-1"°	2 1 Extra $\frac{1}{2}$"° straight	1$\frac{1}{2}$"			2	
b5	20"	33"	30$\frac{1}{2}$"	31$\frac{1}{2}$"	4-$\frac{3}{4}$"°	2	1$\frac{1}{2}$"				
g1	12"	33"	29$\frac{1}{2}$"	29$\frac{1}{2}$"	8-$\frac{7}{8}$"° two rows	4	2$\frac{1}{2}$"	2"		2	
g2	12"	35"	31$\frac{1}{2}$"	31$\frac{1}{2}$"	8-1"° " "	4	2$\frac{1}{2}$"	2"		4 at inner support 2 " outer "	
g3	20"	33"	30$\frac{1}{2}$"	30$\frac{1}{2}$"	4-1"° Rods	2	2$\frac{1}{2}$"				

presented in a somewhat similar fashion to the manner shown in Art. 43. Footing data should also be presented in some suitable form. The same letters and figures should be employed to designate the different members as is shown in the text.

Problems

26. Submit all data for beams, girders, columns, and footings, exclusive of those surrounding the stair and elevator shafts. Use only plain round rods of medium steel and employ the working stresses recommended by the Joint Committee. Consider the wall beams and wall girders as rectangular in section.
27. Make complete details of elevator shaft and pent house, and submit drawings for approval. All beam, girder, and other data should be submitted in the regular forms.
28. Make similar details, etc., for stair shaft.
29. Send in completed drawings of entire building showing all steel reinforcement, dowels, etc. All structural parts indicated in the suggested drawings should be carefully detailed.

CHAPTER XVII

EXAMPLE OF A BUILDING DESIGN INCLUDING THE SPECIFICATIONS

The building erected for the Frank E. Davis Fish Co., Gloucester, Mass., during the years 1910–1911 contains many special features in design. The writer believes that the plans and specifications for this building are sufficiently instructive to include

Courtesy of Densmore & LeClear.

FIG. 326.—Front view of Frank E. Davis Fish Co.'s building, Gloucester, Mass.

the greater part of them in this text. A study of the material presented should give the student a better grasp of the subject of reinforced-concrete design and construction, and the details incidental thereto. The floor design is, of course, very simple but the student should need no further instruction along this line.

The plans and specifications are the work of Densmore and LeClear, Engineers and Architects, Boston, Mass., who have kindly permitted the use of same in this text. It was found impossible to include all the notes given on the original drawings but all those of special importance are shown. The third-floor plan is not given as the second and third floors are similar in nearly every particular. The sections on "Plumbing" and "Gas

Courtesy of Densmore & LeClear.

Fig. 327.—Rear view of Frank E. Davis Fish Co.'s building, Gloucester, Mass.

Piping" have been omitted from the specifications. Front and rear views of the building are presented in Figs. 326 and 327 respectively. A method of estimating the concrete work is shown in Chapter XXVII.

SPECIFICATIONS IN GENERAL

Materials and Labor to be Furnished by Contractor:

1. The Contractor shall furnish all materials and labor, in connection with the work included in this contract, which are mentioned, specified, or indicated, in the specifications or on the drawings, or which are necessary for the proper completion of the work.

Outline:

1. The building is to be of reinforced-concrete construction throughout.

Drawings:

1. The drawings of the work, which form a part of this contract are:

2. Detail drawings, if necessary, will be furnished by the Engineers, and shall be followed in preference to general drawings.

3. Figured dimensions shall be followed in preference to scale dimensions. All dimensions shall be verified at the work where possible.

4. The Contractor shall provide working drawings of any part of the work, upon request by the Engineers.

5. (a) The Engineers will provide the Contractor with two blue prints of each drawing and with one copy of the specifications in addition to the signed copy.

(b) The Contractor is to provide at his own expense any additional blue prints or copies of specifications which may be required by him or by his sub-contractors.

Application of Contract Form and "In General":

1. The Contractor is to see that all sub-contractors read the terms and conditions of the Contract Form and of "In General," all of which apply to all sub-contractors as well as to the Contractor.

Specimens:

1. The Contractor shall submit to the Engineers, upon request, for their approval, samples of any materials which are specified for or intended to be used in the performance of the work, and all materials furnished are to be equal in every respect to the samples approved by the Engineers.

Cutting, Patching, etc.:

1. The Contractor is to do all cutting, drilling, patching, etc., necessary for any of the work included in this contract, or is to make the necessary arrangements for the doing of such work by his sub-contractors.

2. The Contractor is to do all cutting, patching, etc., for the electrical work, steam-fitting, sprinkler and other fire-protection work, not included in this contract, except that small drilling for expansion bolts, etc., will be done by the electrician, steamfitter, etc.

Temporary Heat:

1. Is not to be provided by the Contractor.

Cleaning: Pointing:

1. All the tools of the Contractor are to be removed from the premises upon completion of the work; all dirt, debris, and waste materials caused in the performance of the work are to be removed from the premises from time to time as may be necessary and when directed by the Engineers.

2. All of the work is to be thoroughly cleaned and swept at completion, and all glass is to be washed, leaving the building clear, clean, and ready for use.

3. The final cleaning is to be done after all contractors, sub-con-

tractors, and mechanics employed upon the building have completed their work.

4. The window frames are to be carefully pointed with cement mortar when the exterior cleaning is being done; the staff beads are to be removed and replaced as specified below under "Carpentry."

5. The spaces under the window sills are to be filled with oakum and, for the last inch, with cement mortar.

Notices, City Inspection, Etc.:

1. The Contractor is to give proper notices to inspectors of city, state, etc., and is to have all parts of the work inspected and approved by them as may be required.

2. If the City requires a city inspector at the work continuously the charges of the City therefore will be borne by the Owner.

Contractor's House:

1. The Contractor is to provide and maintain, during the entire progress of the work, a suitable house with necessary tables, racks, etc., for the storing, handling and inspection of the drawings and specifications.

2. It is to be properly heated and lighted, at the expense of the Contractor, and is to be used by all sub-contractors and by all other mechanics employed on the building, whether or not their work is included in this contract.

Protection of Public:

1. The Contractor is to provide and maintain all necessary guards, fences, lights, signs, etc., for the proper protection of the public.

Water:

1. The Contractor is to provide abundant clean water for all parts of the work, making all necessary arrangements with the city, paying all fees, etc.

Building Risk and Watchman:

1. The Contractor is to assume all responsibility for damage to the building and to the materials to be used in its construction, and is to provide a watchman at night and on Sundays and holidays, when the condition of the building requires it.

Lines, Levels, Batter Boards:

1. The Contractor is to employ an experienced engineer to establish and from time to time to check, by instrument, all lines, levels, and grades of the building, and alignment of piers, columns, etc.

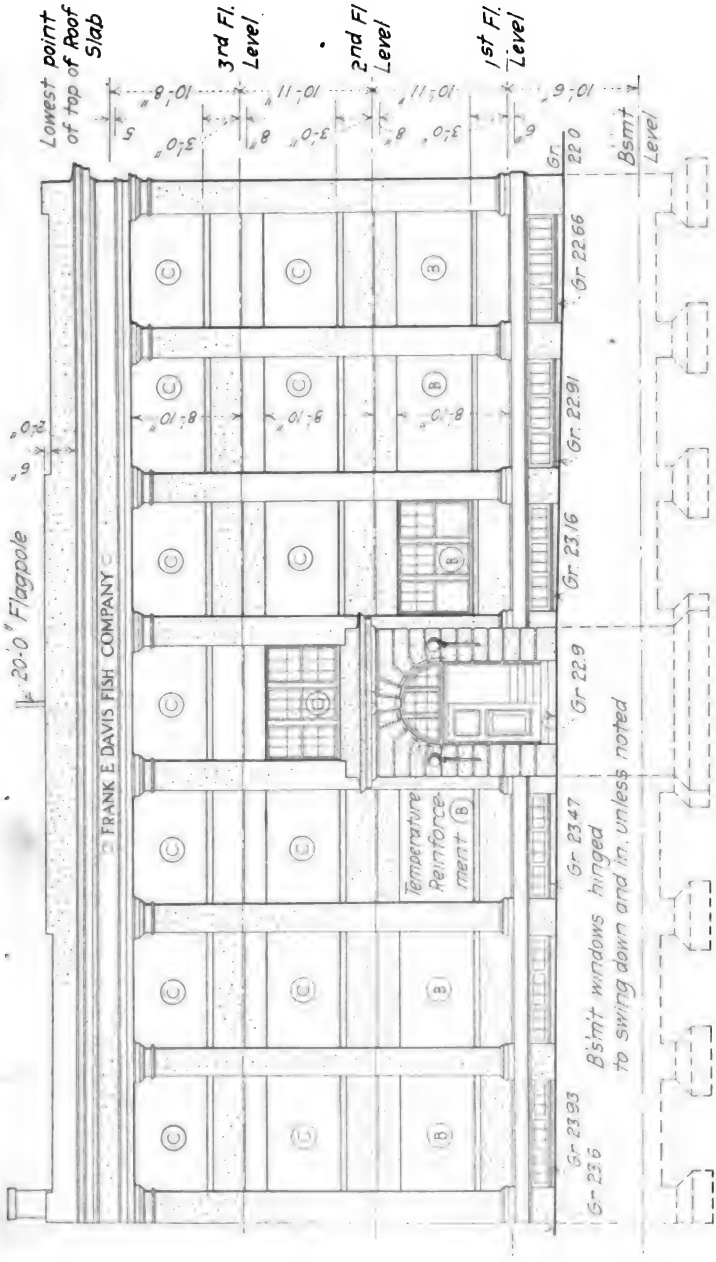
2. Batter boards are to be put up at proper distances from the building and the lines, levels, and grades are to be plainly marked on them.

3. All grades marked on the drawings refer to the mean low water as based on the assumption that the capping of the Davis Wharf is 14.5 ft. above mean low water.

Drains, Pipes, Wires, Etc.:

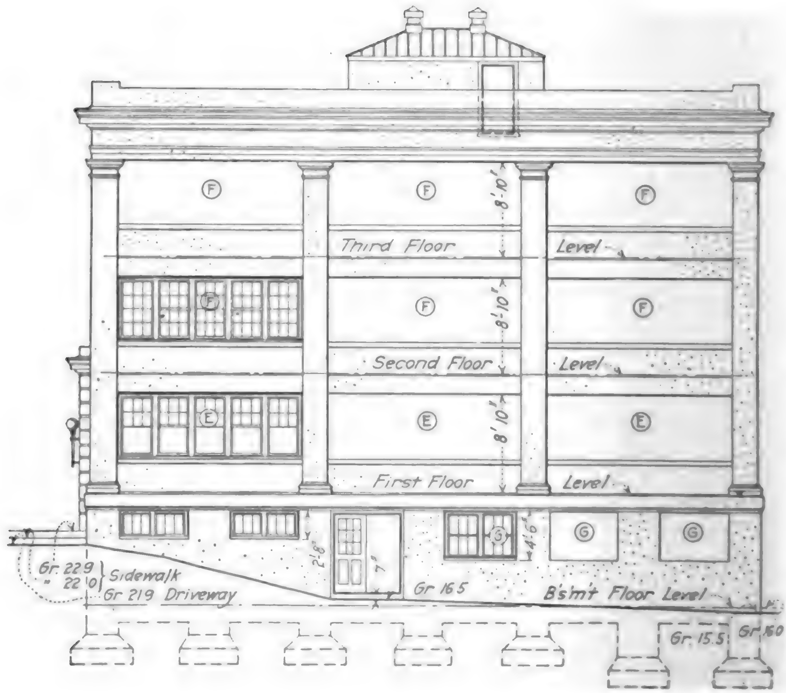
1. All existing drains, pipes, wires, telephone and telegraph poles, etc., in the way of or disturbed by the progress of the work are to be removed, replaced, extended, or repaired, as circumstances prove desirable.

PLATE XXIV



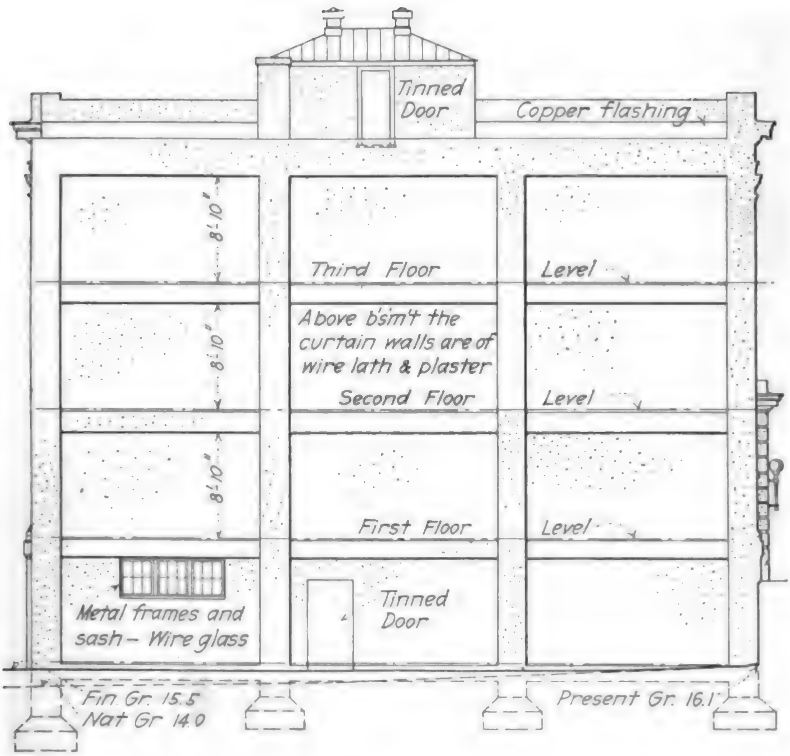
Frank E. Davis Fish Co.'s building—Front elevation.

PLATE XXV



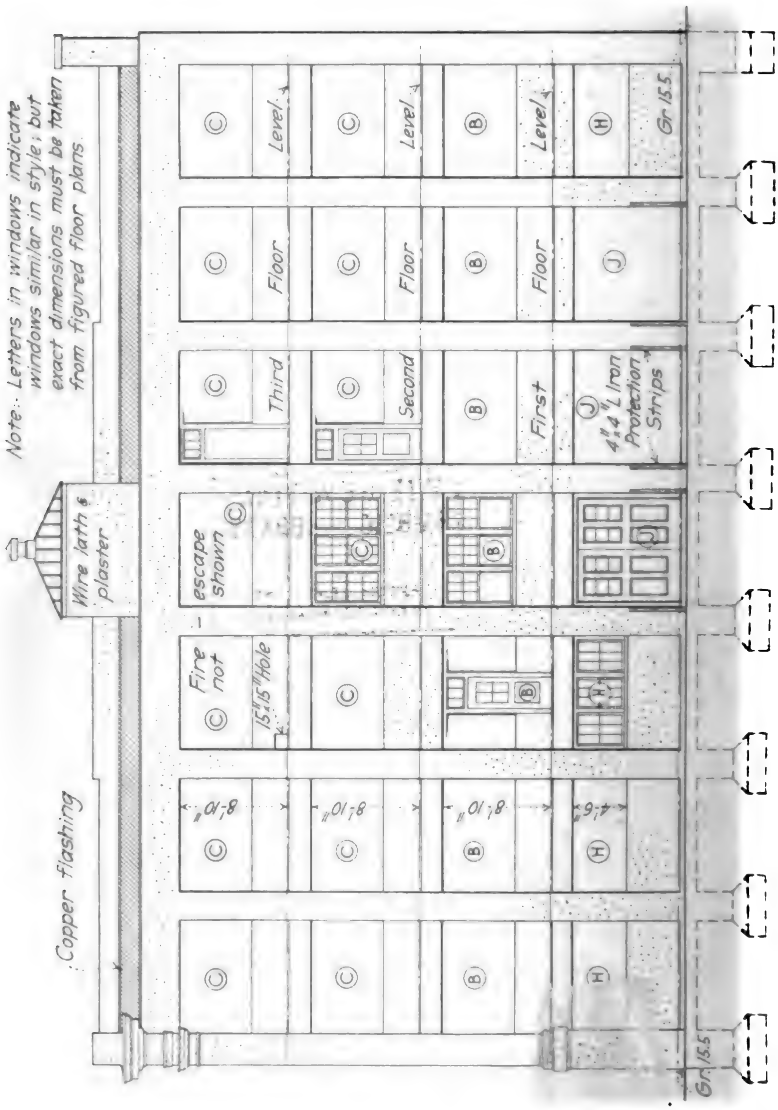
Frank E. Davis Fish Co.'s building—West elevation.

PLATE XXVI

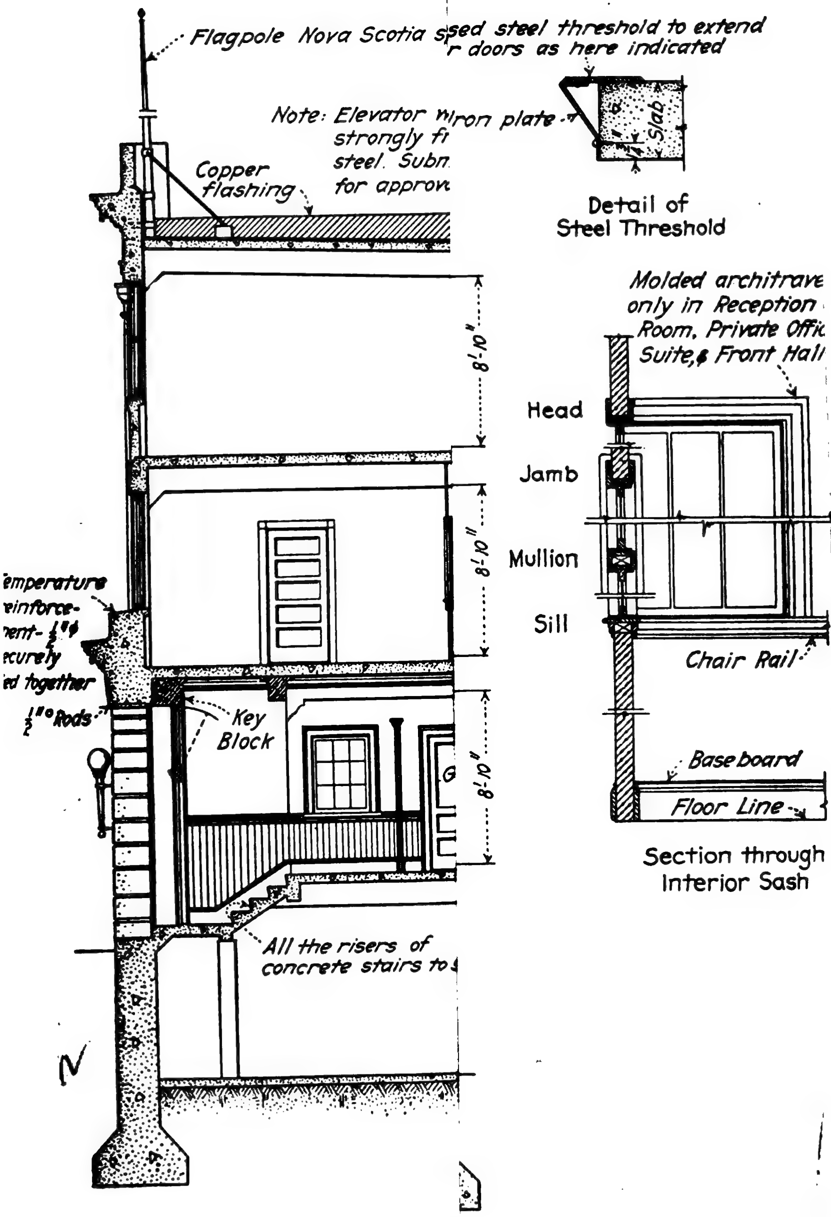


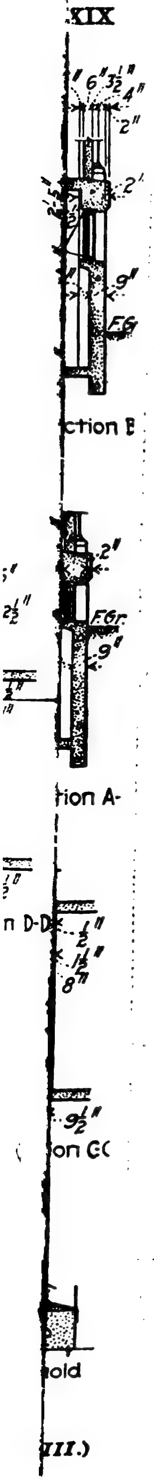
Frank E. Davis Fish Co.'s building—East elevation.

PLATE XXVII



Frank E. Davis Fish Co.'s building—Rear elevation.


Flagpole
Nova Scotia s
sed steel threshold to extend
r doors as here indicated
Note: Elevator w
strongly fr
steel. Subm
for approv
Iron plate
Slab
Copper flashing
Detail of Steel Threshold
Molded architrave only in Reception Room, Private Offic Suite, & Front Hall
Head
Jamb
Mullion
Sill
Chair Rail
Baseboard
Floor Line
Section through Interior Sash
8'-10"
8'-10"
8'-10"
emperature einforce-ment-½"ϕ ecurely ed together
½"ϕ Rods
Key Block
All the risers of concrete stairs to


XIX
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III.)

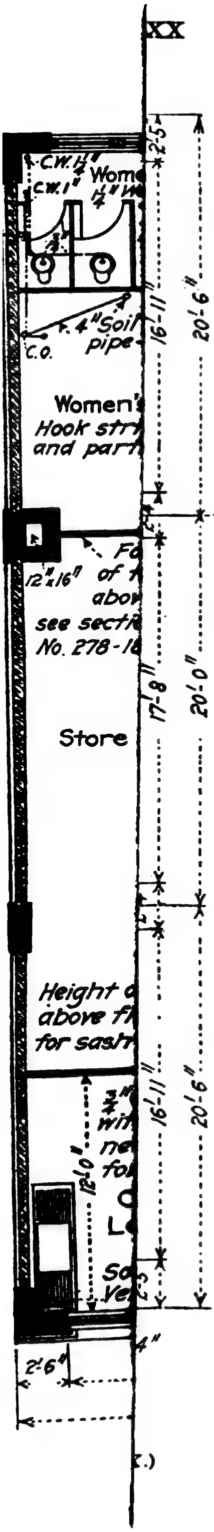
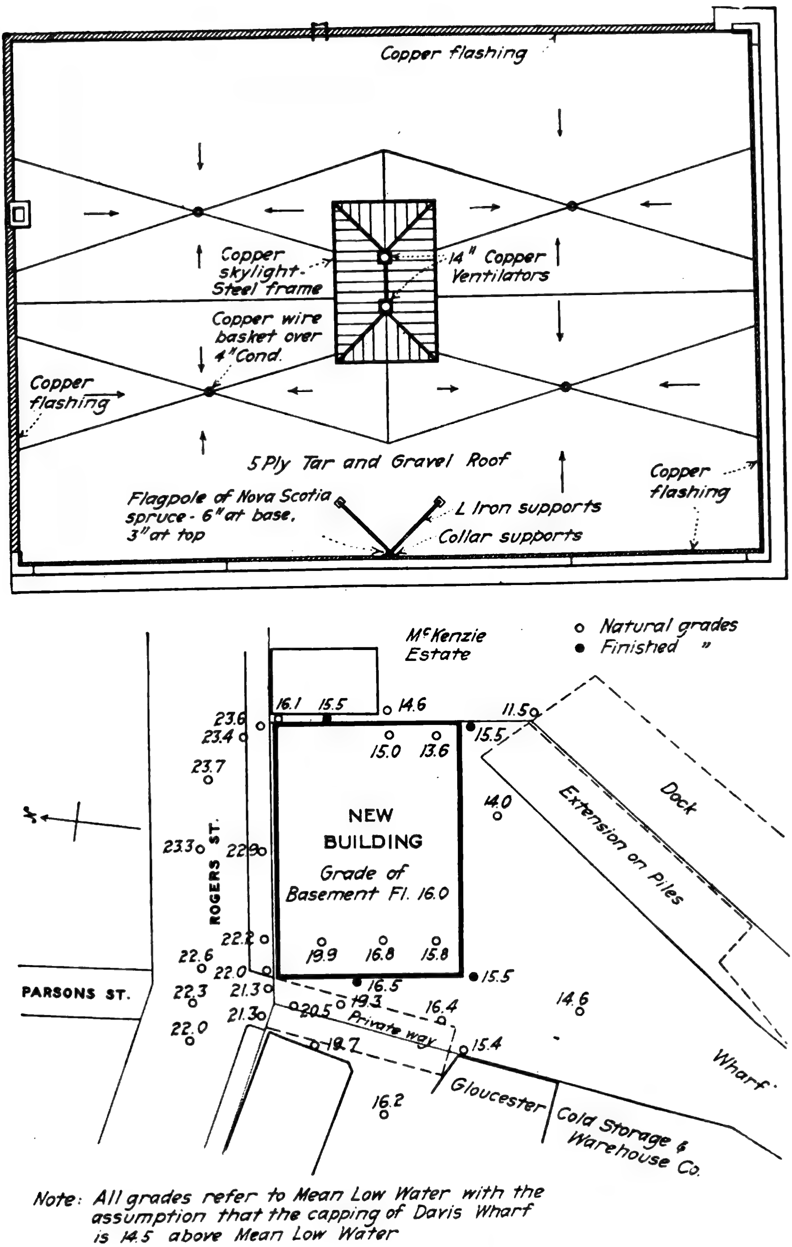
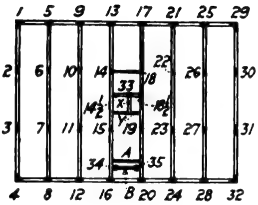

XX
C.W. 1 1/4"
Wome
C.W. 1"
4" Soil pipe
C.O.
Women's
Hook str
and part
12"x16"
see secti
No. 278-16
Store
Height a
above fl
for sash
2'-5"
16'-11"
20'-6"
17'-8"
20'-0"
16'-11"
20'-6"
12'-0"
4"
2'-6"

PLATE XXXII

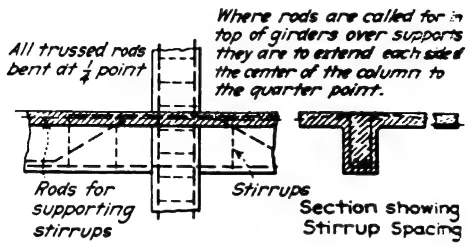


Frank E. Davis Fish Co.'s building—Roof and location plans.

PLATE XXXIII.



Framing Plan



Typical Scheme of Reinforcement of T-Girders

ROOF LEVEL GIRDER DATA

Designation	Size D	Size W	Reinforcement
All interior girders except as otherwise noted	20″	10″	2-1″° upper row trussed 3-⅞″° lower " " 6-⅜″° stirrups each end, spaced 6-9-11-12-12-12
1-2, 2-3, 3,4	See Plan	11½″	3-⅞″° Rods one row 3-⅞″° " on top over supports - 4-⅜″° stirrups each end spaced 6-9-12-12
29-30, 30-31, 31-32	See Plans	See Plan	See special detail
Rear wall 1-5 to 25-29 Inclusive	See Plan	11½″	2-⅝″° Rods one row
Front wall 4-8 to 28-32 Inclusive	See Plan	See Plan	3-¾″° Rods one row
14-18 and 14½-18½	17″	8″	2-¾″° Rods one row 3-⅜″° Stirrups each end spaced 8-10-12
Y	9½″	6″	2-½″° Rods one row

FIRST FLOOR LEVEL GIRDER DATA

Designation	Size D	Size W	Reinforcement
All interior girders except as otherwise noted	21″	18″	2-1⅛″° upper row trussed 3-1⅛″° lower " 5-½″° Stirrups each end spaced 8-12-12-12-12
A X	12″ 9″	6″ 6″	2-⅜″° Rods one row 2-⅝″° " " "
Y	13½″	12″	2-¾″° 2-1½″° one row 2-⅜ stirrups each end spaced 6″-10″
14-15, 18-19	21″	18″	2-1″° upper row truss'd 3-¾″° lower " Stirrups as for typical girder

THIRD FLOOR LEVEL GIRDER DATA

Designation	Size D	Size W	Reinforcement
All interior girders except as otherwise noted	23″	18″	2-1½″° Rods upper row trussed 3-1⅛″° Rods lower row 10-½″° stirrups each end spaced [illegible]
1-2, 2-3, 3-4, 29-30, 30-31, 31-32	See Plan	11″	2-1″° upper row trussed 1-1″° lower " 2-⅞″° " " 5-½″° stirrups each end spaced 6-8-12-[illegible]
Rear wall 1-5 25-29 Inclusive Front wall 4-8 28-32, Incl.	See Plan	11″	2-½″° Rods one row
14-15	23″	18″	2-1⅛″° Rods upper row trussed 3-⅞″ lower row trussed 6-½″° Stirrups each end spaced [illegible]
[illegible]	23″	18″	2-1¼″° upper row trussed 3-⅞″° Rods lower row Stirrups as for 14-15

FIRST FLOOR LEVEL-GIRDER DATA

B	9½″	6″	2-⅝″° one row 2-⅜″ stirrups spaced [illegible]
Front wall 4-8, 28-32 Inclusive Rear wall 1-5, 25-29 Inclusive	See Plan	See Plan	2-½″° Rods one row
29-30, 30-31, 31-32	See Plan	17½″	2-⅝″° Rods one row
1-2, 2-3, 3-4	See Plan	11″	2-1″° upper row trussed 3-⅞″° lower " 5-½″° stirrups each end spaced 8-10-12-12-12

Frank E. Davis Fish Co.'s building—Framing plan and girder data.

PLATE XXXIV

SECOND FLOOR LEVEL GIRDER DATA

Designation	Size D W	Reinforcement
All girders like	3rd Floor	except as below noted
16-20	See Plan	3-$\frac{3}{4}$"° Rods one row See details for Temperature Reinforcement

Note: D = Depth of girders (Top of slab to bott. of stem)
W = Width " " (Width of stem)

STAIR RUNS

Location	Slab	Reinforcement	Spacing of Rods
B'sm't to 1st Fl	$7\frac{1}{2}$"	$\frac{1}{2}$"° Rods	4"
Front stairs Entrance to Vestibule	4"	$\frac{3}{8}$"° "	10"
Land'g Foot of Stairs Ent. to Vestibule	4"	$\frac{3}{8}$"° "	10"

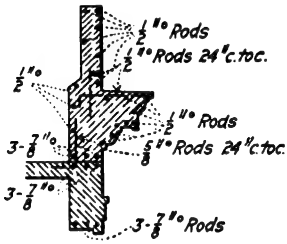
COLUMN DATA

Column Numbers	Basement	1st Floor	2nd Floor	3rd Floor
	Column size and reinforcement			
6-7-10-11-15-18 19-22-23-26-27	18"x 22" 4-$1\frac{1}{4}$"° Rods	18"x 18" 4-$1\frac{1}{4}$"° Rods	18"x 18" 4-1"° Rods	12"x 12" 4-$\frac{1}{2}$"° Rods
8-12-16 20-24-28 30-31	2'-2" 2'-4" 1'-4" 6-$\frac{7}{8}$"° Rods	12"x 26" 6-$\frac{7}{8}$"° Rods	12"x 26" 6-$\frac{3}{4}$"° Rods	12"x 26" 6-$\frac{1}{2}$"° Rods
2-3-5 9 13-17 21 25	2'-2" 2'-4" 12" 6-$\frac{7}{8}$"° Rods	12"x 26" 6-$\frac{7}{8}$"° Rods	12"x 26" 6-$\frac{3}{4}$"° Rods	12"x 26" 6-$\frac{1}{2}$"° Rods
14	18"x 22" 4-$1\frac{1}{4}$"° Rods	18"x 18" 4-$1\frac{1}{4}$"° Rods	18"x 18" 4-1"° Rods	12"x 15" 4-$\frac{1}{2}$"° Rods
1 4 29-32	See Plans 8-$\frac{7}{8}$"° Rods	See Plans 8-$\frac{7}{8}$"° Rods	See Plans 8-$\frac{3}{4}$"° Rods	See Plans 6-$\frac{1}{2}$"° Rods

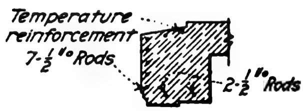
Note:- Sizes of Cols. where no sketch is shown are net.

SLABS

Location	Thickness	Steel	Spacing	Notes
Roof	5"	$\frac{1}{2}$"°	7" c. to c.	These rods to be near top of slab after crossing girder
2nd & 3rd Fl.	8"	$\frac{5}{8}$"°	$6\frac{1}{4}$" c. to c.	
1st Floor	6"	$\frac{1}{2}$"°	$5\frac{3}{4}$" c. to c.	
Cantilever slab each side Front vestibule stairs	6"	$\frac{1}{2}$"°	$5\frac{3}{4}$" c. to c.	
1st Floor under start of stair to second	4"	$\frac{3}{8}$"°	5" c. to c.	
Cantilever slab back of Elevator 2nd · 3rd	8"	$\frac{5}{8}$"°	$6\frac{1}{4}$" c. to c.	



Section through Cornice & Roof Girder 29-30, 30-31, 31-32.



Section through Water Table

TEMPERATURE REINFORCEMENT

All concrete curtain walls, including b'sm't curtain walls on south, east and west sides, to have temperature reinforcement as follows, $\frac{1}{2}$"° Rods 12" c. to c. horizontal, 24" c. to c. vertical. All temp. reinforcement to be placed near outside face. All 1st, 2nd, & 3rd Fl. slabs and floor of Cleaning Rm. to have surfaces reinforced with $\frac{1}{4}$"° rods 18" c. to c. at rt. ∠s to general reinforcement where such occurs. All temp. rein. to be securely tied in place. Roof wall girder south and east sides to have 4-$\frac{1}{2}$"° rods, approx. 8" c. to c. near outside face. For temp. reinforcement in front entrance porch see Plates XXIV & XXVIII. See also miscellaneous sections.

Note:- Reinforcem't vert. $\frac{1}{2}$"° @ 12" Reinforcement Hor. $\frac{5}{8}$"° rods spaced from top down to Gr. 15.90 as follows 12"-12-12-12-8-8-8-6-6" all placed near back of wall, one extra rod at bottom near outside. Exterior face of retaining wall to have same temperature reinforcement as curtain wall.

Frank E. Davis Fish Co.'s building—Column, stair, and other data.

2. Where such materials are the property of the city or of public service corporations the Contractor is to make all necessary arrangements with the city or corporations.

Prepayment of Express and Freight Charges:

1. The Contractor is to see to it that all materials are shipped to the building prepaid, as the Owner will not advance payment for any express, freight, or similar charges.

Inspection of Premises:

1. The Contractor is to visit the site of the building and make himself thoroughly familiar with the premises, the nature of the soil, and the conditions under which the work is to be performed, before submitting a figure; he is also to see that all sub-contractors similarly investigate the conditions under which the work is to be done, as may be necessary.

Templates, Models, Etc.:

1. The Contractor is to provide all templates or models which may be required.

2. Models are provided for under the heading "Allowances" below.

Closing in and Protection of Work:

1. Temporary doors or screens for all openings in the building are to be furnished to keep out the weather, when directed by the Engineers.

2. The building is to be kept locked when no work is being done in it, beginning at the earliest time practicable and continuing until the work is completed.

3. The stair treads are to be covered with temporary boards and all other parts of the work are to be protected from injury as may be found necessary; any part of the work damaged in any manner is to be replaced at completion of the work at the expense of the Contractor.

Guarantee:

1. The Contractor is to make good any defects, settlements, shrinkage, injurious cracks or other faults arising from defective or improper workmanship or materials which appear within one year from the date of the completion of the work included in this contract.

2. The Contractor is also to guarantee the roof, as specified below under that heading.

EXCAVATING; REFILLING; GRADING

Removal of Present Building:

1. The present building entire, excepting the front basement wall will be removed down to grade 15.5 by the Owner.

2. The front basement wall (which is also a retaining wall for the street) and all construction below grade 15.5 as necessary are to be removed by the Contractor.

3. All necessary shoring is to be included by the Contractor.

Excavating:

1. Is to be done for the entire work to required grades, all referred to the "base" noted on page 434.

2. Excavations are to be made wide enough to permit of the proper installation of forms and the mopping of and filling in and around exterior walls. (The filling in and mopping are specified under "Refilling" and "Concrete Work" respectively.)

3. All shoring and sheet piling, etc., necessary for completing the excavating or for maintaining the excavations for the proper completion of any of the work of this contract, are to be included, and are to be properly protected until all refilling has been completed.

4. All excavations are to be kept free from dirt, rubbish, or other materials, and all water is to be pumped from the excavations as may be necessary for the proper completion of the work included in this contract.

5. Excavated material is to be used for refilling and grading, except rubbish, sticks, etc., which are to be piled on the premises where directed; surplus excavated material, if any, is to be piled on the premises, where directed.

Refilling:

1. All excavations are to be filled to the grades required.

2. (a) All excavations around exterior walls are to be filled in from a point 1 ft. below the level of the bottom of the basement floor slab to finished grade for a distance not less than 18 in. from the walls.

(b) The filling around exterior walls is to be clear gravel or crushed stone up to a level 18 in. below finished grade.

(c) The filling around exterior walls from a level 18 in. below up to finished grade is to be good clean earth.

3. (a) All filling is to be thoroughly rammed and watered so as to pack it thoroughly; (b) all filling under floors is, in addition, to be rolled and cross rolled in layers, made smooth and level, and carried full to the under side of the floor slab; this work is to be done as soon as practicable so as to be as solid as possible when the basement floors are put down.

Grading:

1. The excavated material is to be used for grading to levels shown on the drawings, all rubbish, sticks, etc., being removed.

2. No new material for grading is to be provided if the excavated material left over after refilling is insufficient.

CONCRETE WORK

In General:

1. All construction is to be of reinforced concrete unless otherwise indicated or specified.

2. All materials and workmanship are to be of the best grade in accordance with the best modern practice, and all work is to be done under the direct superintendence of a capable foreman, experienced in reinforced-concrete construction.

Materials:

Cement:

1. Is to be high-grade, well-seasoned, American cement.

2. Is to be delivered in suitable packages with the brand and name of the manufacturer clearly marked thereon.

3. (a) Is to be stored in a suitable water-tight building and kept free from moisture; (b) is to be so stored that shipments will be used in the order of their delivery; (c) no cement of any shipment is to be used sooner than 12 days after delivery unless samples have first been tested and have passed the requirements for the "7 days' test."

4. (a) Each shipment is to be tested by an expert selected by the Engineers; the expense is provided for under "Allowances."
(b) The tests are to be made in accordance with the methods proposed by the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, in their report of January 21, 1903, with all subsequent amendments.
(c) Samples must show results in accordance with the requirements of the American Society for Testing Materials, except those for tensile strength, the requirements for which shall be:

Neat Cement

Age	Tensile strength
24 hours in moist air	175 lb. per square foot
7 days (1 day in moist air, 6 days in water)	550 lb. per square foot
28 days (1 day in moist air, 27 days in water)	650 lb. per square foot

One Part Cement, Three Parts Sand

7 days (1 day in moist air, 6 days in water)	200 lb. per square foot
28 days (1 day in moist air, 27 days in water)	275 lb. per square foot

The five-hour steam test may be replaced by a four-hour boiling test.
(d) The contractor is to provide a competent man to aid in taking samples, when requested by the Engineers.
(e) The tests shall include the determination of specific gravity, fineness, time of setting, tensile strength, constancy of volume, and percentage of sulphuric acid and magnesia, and samples must show results in accordance with the requirements of the American Society for Testing Materials.
(f) The sand used in the tests is to be "Ottawa" sand.

Sand:

1. Is to be coarse, sharp, and clean, free from clay, dirt, loam or other foreign substances.

2. Is to be furnished as required for making "cement-sand" samples, for testing.

3. Is to be tested by an expert selected by the Engineers, and when mixed with the cement in the proportions of one part of cement to three

parts of sand must give tensile strength not less then 70 per cent of the tensile strength specified above for the cement mixed with "Ottawa" sand.

4. The expense of the testing is provided for under "Allowances."

Stone:

1. Is to be clean, sharp granite, without any slate, shale, or limestone, and free from all foreign substances.

2. Is to be screened clear of all dust and sand and of all stone which will pass through a ¼-in. mesh screen, except stone for granolithic or for other special finish, which is to be screened of dust, but is to pass through ½-in. mesh or small screen, as required.

3. Is to be of graduated size in each mixture; the maximum sizes for different parts of the work are specified under "Mixtures" below.

Cinders:

1. Are to be clean, free from ashes or other foreign matter.

Reinforcement:

1. Is to be open hearth or Bessemer "Medium" steel, in the form of plain round rods, and having the properties required by the "Manufacturers Standard Specifications."

2. (a) Tests are to be made by an expert selected by the Engineers; (b) any shipment of steel is to be rejected if the tests indicate that the steel is not up to the specifications; (c) the expense of testing is provided for under "Allowances."

3. The sub-contractor furnishing the steel is to be approved by the engineers before the contract is signed.

4. (a) "Deformed" bars, of same area as the round rods specified, may be substituted subject to the approval of the Engineers.

(b) In the case of such substitution the properties of the steel must be approved by the Engineers and filled in below before the contract is signed, and the same requirements regarding tests shall hold as specified for round rods.

Mixtures:

Determination of Proportions:

1. (a) Bottomless boxes, of dimensions approved by the Engineers, are to be made for determining the proportions of sand and stone, and are to be carefully preserved throughout the progress of the work.

(b) All barrows used for sand, stone, and cement are to be of the same type and size and are to be carefully calibrated for determining proportions, but must be tested from time to time by comparison with the boxes, when required by the Engineers.

2. Proportions of cement are to be determined by weight, on the basis of 1 cu. ft. weighing 100 lb. or a standard bag of 95 lb. containing 95/100 cu. ft.

3. All mixtures are to be made with a suitable mechanical mixer.

Consistency of Mixtures:

1. The mix is to be "wet," the exact consistency being determined at the building.

Stone Concrete:

1. Is to be mixed in the proportions of one part of cement—2½ parts

of sand—5 parts of stone, of graduated sizes, the maximum of which will pass a 2½-in. mesh for

(a) Footings.

(b) Basement floor except for dressing.

2. Is to be mixed in the proportions of 1 part of cement, 2 parts of sand, 4 parts of stone, of graduated sizes, the maximum of which will pass a 1-in. mesh, for

(a) All parts of the work other than specified above under (1), except granolithic, and except finish work specified below.

3. Is to be mixed in the proportions specified under (2) above, but with stone, the maximum size of which will pass a ½-in. mesh, for all molded work.

4. The exposed exterior surface of the main front entrance porch is to be made with concrete composed of the same mix specified under (2) above, except that the stone is to be pass a ½-in. mesh and is to be of such color as to give a uniform effect.

5. (a) A sample of the special finish concrete specified under (4) above, not less than 1 ft. square, is to be submitted after the contract is awarded.

6. The name panel on the front elevation is to be made with very small stone so that the surface shall be suitable for carving.

Cinder Concrete:

1. Is to be mixed in the proportions of 1 part of cement, 4 parts of sand, 8 parts of cinders.

Granolithic:

1. Is to be mixed in the proportions of 1 part of cement, 1 part sand, and 1 part crushed stone, maximum size to pass through ½-in. mesh.

2. Is to be laid monolithic with construction work and in the best manner.

Cement Mortar and Grout:

1. Is to be mixed in the proportion of 1 part of cement and 2 parts of sand.

Forms and Molds:

1. (a) Are to be provided for all parts of the work except for basement floor and for bottom of footings.

(b) Where forms are not used the fill is to be made true and smooth.

2. Are to be made of green material, free from shakes, loose knots, warping, splinters, etc., where adjacent to concrete.

3. Are to be accurately put together and nicely fitted, with close joints.

4. Are to be of sufficient thickness, with sufficient ties and bracing, to prevent all deflection either from the weight of the concrete, men, or machinery, or from the ramming of the concrete or from other work going on.

5. Are to be securely supported at frequent intervals to prevent any change in form; the supports are to be carried down to solid work and must in no case rely on recently erected construction; the method of support is to be subjected to the approval of the Engineers.

6. Are to be installed so that columns, girders, piers, beams, slabs, etc., shall be plumb, level and true.

7. Are to be nicely dressed on the sides against the concrete for all construction where the concrete will be exposed to view after completion, either interior or exterior.

8. Are to be properly greased or oiled on the surface in contact with the concrete.

9. Are to be so constructed as to be readily taken down without damaging the concrete.

10. Are to be throughly cleaned and swept just before concrete is dumped, suitable trap doors being installed at the bottom of all column forms for cleaning and inspection.

11. Are to be left in place until the concrete has set sufficiently, as approved by the Engineers.

12. All right-angle corners on girders and beams and interior columns are to be prevented by installing triangular strips in the forms.

13. All forms for rustications, moldings, etc., are to be carefully placed and secured and are to be made to detail of true and uniform cross-section throughout.

Construction Work:

Placing Concrete:

1. The concrete is to be placed carefully, so as not to separate the aggregate.

2. Each layer is to be thoroughly tamped and spaded so as to bring the water to the surface, to completely fill the forms and to pack solidly around all reinforcement.

3. Wherever the concrete is to be exposed it is to be pried away from the forms before tamping so that the mortar will be brought to the surface.

4. Column tops, girders, and beams are to be poured as monoliths with the floors they support.

5. The columns are to be poured from the floor levels to the bottoms of the column tops, girders, or beams and are then to be allowed to take their shrinkage before the tops, girders, or beams are poured.

6. (a) Joints due to temporary stopping of work are to be made where the least harm to strength and appearance will be produced by the lack of a perfect monolithic construction.

(b) Joints in floor slabs, beams and girders, are to be avoided as far as possible, but when necessary are, in general, to be made in the middle of the bays.

(c) All locations of joints are to be approved by the Engineers.

(d) Where joints occur, the surfaces are to be made approximately true by stopping the concrete against temporary forms.

(e) No concrete is to be laid in freezing weather; any work which has been subjected to a temperature of less than 30° F. within 48 hours after being placed for a sufficient time, in the opinion of the Engineers, to cause injury to it, is to be removed and replaced.

7. All concrete is to be kept thoroughly wet down in warm weather

for one week after being placed, and, where feasible, is to be protected from the sun.

8. All wooden and metal blocks, sockets, anchors, bolts, pipe sleeves, etc., etc., are to be built in, care being taken that the concrete is closely packed around them.

9. After the forms are removed all necessary repairs are to be made with cement mortar to produce a smooth flush surface.

Placing Reinforcement:

1. All concrete work is to be reinforced with steel, as indicated on the drawings where details of reinforcement are given, and as directed by the Engineers where details of reinforcement are not given.

2. No reinforcing steel is to be painted or oiled; but is to be thoroughly cleaned and freed from rust scales before being set in place.

3. All reinforcing steel is to be carefully and accurately bent to the exact shapes required and is to be accurately placed and securely tied in place so as to insure the preservation of its proper shape and position after the concrete is placed and tamped.

4. All reinforcing steel for girders and beams is to be made up in "unit frames" before being placed.

5. Rods are to have a lap of 40 diameters, as indicated on the drawings, or are to be secured to develop the strength of the rod in manner approved by the Engineers.

6. All column and pier reinforcing rods are to be properly joined and secured, and are to be tied together with steel wire every 12 in. in height.

7. All reinforcement, except "temperature reinforcement," is to be so placed the it shall be covered by concrete not less than $\frac{3}{4}$ in. in slabs, nor less than $1\frac{1}{2}$ in. in girders, beams, and columns.

8. All reinforcing steel is to be inspected and approved in place by the Engineers before it is covered with concrete, and no concrete is to be placed until the approval of the reinforcement by the Engineers has been obtained.

Floors:

1. Are all to be stone concrete with granolithic finish, except where otherwise specified below.

2. Are all to be lined in squares.

3. All granolithic is to be of the highest grade of workmanship.

4. The floors are to pitch to drain where so indicated; elsewhere they are to be level.

6. The floor in the cleaning room, in the basement, is to have sanitary base turned up all around the room, with 1-in. radius, ready to receive tiling on walls.

7. The barrel cleaning pit in basement is to be built essentially as indicated.

Roof:

1. Is to be finished smooth as necessary for installing the roofing specified under "Roofing and Sheet Metal Work."

2. (a) Is to pitch to drain, as indicated; (b) the pitch is to be obtained

by pitching the roof slabs; (c) cinder concrete crickets are to be installed as required.

Stairs and Landings:

1. The main stairs and landings from first to third floors, except the landing at floor levels, are to be of iron as specified below, all other stairs and landings are to be of concrete.

2. (a) Concrete stairs are to have granolithic finish.

(b) Are to be reinforced in accordance with details or directions to be provided by the Engineers.

3. All sockets for railings, etc., are to be built in place, the sockets being provided as specified under "Steel and Iron Work."

4. The front steps and landings are to be made of concrete with a small percentage of hydrated lime, as specified under "Waterproofing."

5. Treads and landings are to be rebated for the safety treads specified under "Steel and Iron Work," and holding sockets for same are to be built in.

Door Thresholds:

1. Are to be, where so marked, of granolithic, properly reinforced.

2. Exterior thresholds or sills are to be made with wash.

3. Elevator door openings are to have steel thresholds securely anchored in place. (The steel is specified under "Steel and Iron Work.")

Parapet Walls, Cornices, and Other Molded Work:

1. Are to be molded in place, essentially as indicated on the drawings, and in strict accordance with detail drawings.

Building in Flashing:

1. All flashing and counterflashing are to be built in or reglets are to be cut to receive same.

2. The flashing is provided for under "Roofing and Sheet Metal Work."

Building in Elevator Steel and Iron Work:

1. All elevator sheave beams, guides, guide supports, beams for supporting machines, bolts, anchors, etc., are specified under "Elevator."

2. The Contractor is to do all cutting and patching for the installation of the above beams, etc.

Waterproofing:

1. All curtain walls above and below grade, the entire parapet wall, the top of main cornice, and the front steps are to be made of concrete to which has been added "Hydrated Lime" in the proportions of ten parts of lime by weight to one hundred parts of cement by weight.

2. The exterior walls are to be thoroughly mopped with hot tar from bottom of retaining wall to finished grade.

Chimney:

1. Is to be built of concrete as indicated with fine clay flue lining for its entire length.

2. (a) Is to have holes for cleanout and entrance of smoke flue where indicated.

(b) Thimbles and cleanout door and frame specified under "Steel and Iron Work" are to be set in place.

Interior Finish (Concrete):

(For other interior finish, see "Lathing and Plastering" and "Carpentry.") (See also "Floors" and "Stairs and Landings" above.)

1. Is to be as it comes from the molds on interior walls, columns, girders and ceilings, all inperfections being carefully filled with cement mortar and all "fins" being removed, leaving the surfaces perfectly flush and true.

2. All surfaces are to be made as smooth as possible without plastering or washing by prying the mixture away from the molds before tamping, bring the mortar to the surface.

Exterior Finish:

(See also, "Stairs and Landings" above.)

1. All imperfections in the entire exterior of the building are to be carefully filled with cement mortar and all "fins" removed.

2. The entire concrete exterior of the building except the east elevation and except the front entrance porch is to be carefully rubbed down and washed with cement wash, made of 1 part of cement, ½ part of hydrated lime and 2 parts of finely screened sand, all mixed dry and then wetted to a proper consistency for application with a white-wash brush.

3. (a) The front entrance porch except its cornice is to be carefully and uniformly picked and then thoroughly washed with acid, the acid being afterward washed off; draft lines, in accordance with directions at the building, are to be rubbed down and washed as specified above.

(b) The cornice is to be rubbed down and washed as specified above.

Building in Sleeves:

1. Wrought iron sleeves with caps screwed on, provided as specified under "Steel and Iron Work," are to be built into the basement wall for the introduction of telephone wires, messenger wires, lighting wires, etc.; the exact locations are to be determined at the building.

2. Wrought iron and galvanized iron sleeves, provided as specified under "Plumbing," are to be built into the construction work for all plumbing pipes; the exact locations are to be determined at the building.

3. Wrought iron and galvanized iron sleeves, provided under another contract, are to be built into the construction work for all steam, return, sprinkler, and similar pipes; the exact locations are to be determined at the building.

4. The Contractor is to be responsible for the proper building in of all sleeves, for securing them firmly in place, and for seeing that they are not disturbed.

5. The Contractor is to finish around all sleeves with cement mortar.

6. All wall sleeves are to finish flush with the finished wall on each side; all floor sleeves are to be flush with the ceiling line and are to extend 2 in. above the floor line.

Building in Electric Conduits:

1. The electric conduit, outlet boxes, etc. are to be installed under

another contract, before the concrete is placed; the boxes are to be firmly bolted to the forms and the conduit secured as well as possible by the Electrician, but the Contractor is to take care to avoid displacing or injuring them.

2. The Contractor is not to place any concrete until the electrical work is ready, and he is to co-operate with the Contractor for electric wiring for the successful completion of the work, is to notify him as far ahead as possible when the conduit must be ready, etc., etc.

Sockets for Hangers:

1. Are provided for under "Steel and Iron Work."

2. Are to be set in place and properly secured to forms so as to be strongly anchored in place for supporting shafting, etc., and so as to be flush with the bottom faces of the members in which they are installed.

3. Are to be installed as follows: (a) four in bottom of each interior girder spaced as directed. (b) In basement ceiling as indicated.

Building in Miscellaneous Iron and Woodwork:

1. All anchors, bolts, flag-pole supports, nailing strips for furring, for baseboards, chair rails, picture molding, etc., etc., necessary for the proper completion of the work are to be built in.

2. The Contractor is to be responsible for the furnishing in time for building in, of all anchors, bolts, blocks, etc. for all parts of the work included in this contract, making whatever arrangements with the various sub-contractors may be necessary.

3. Anchors, bolts, blocks, etc. will be provided under other contracts for supporting steam and other piping and apparatus, but are to be built in by the Contractor.

Load Test:

1. Is to be applied to two bays on the second floor, selected by the Engineers.

2. The test load is to be uniform over the area tested, at 300 lb. per square foot.

3. The load is to be obtained by uniform units which are to be carefully weighed.

4. The Contractor, in the presence of the Engineers, is to measure the deflections of slabs and girders, using suitable instruments, at the following periods:

(a) Just before applying the load;
(b) Twice during the application of the load;
(c) Immediately after applying load;
(d) Twenty-four hours after applying load;
(e) Seven days after applying load.

5. No loads are to be applied until sixty (60) days after concrete is placed.

LATHING AND PLASTERING

Partitions:

In General:

1. The interior partitions are to be built of channel irons with expanded metal lathing and plaster, being 2 in. or 4 in. thick as marked.

Studding:

1. (a) The 2-in. partitions are to have 1-in. channel studding run vertically, 12 in. on centers; (b) the 4-in. partitions are to be made with two rows of $\frac{3}{4}$-in. channels 12 in. on centers; (c) all of the studding is to be strongly secured and braced as necessary for properly supporting the partitions, door frames, window frames, etc.

Framing:

1. The iron frames for tinned doors and rolling steel doors are specified to be provided under "Steel and Iron Work," but the Contractor is to set them in place and secure them.

2. Wooden frames are specified to be provided and erected by the Carpenter, but the Contractor is to provide and erect all angle irons and channel irons necessary for supporting the stud frames.

3. All doors, windows, finish, etc., are specified under "Carpentry," "Sheet Metal Work," but the Contractor is to provide and erect all necessary angles, braces, etc., for the support and attachment of the window frames, etc.

4. All grounds for baseboards, chair rails, picture moldings, hook strips, and other attachments, are to be securely attached, the woodwork being specified under "Carpentry."

5. Metal corner beads are to be used at all external corners.

Wire Lathing of Partitions:

1. Is to be applied to one side of 2-in. partitions and to both sides of all wider partitions, and is to be firmly secured.

2. Is to be carried around columns in plaster partitions, as indicated, with all necessary metal furring, and similarly around columns in cleaning room for furring in of soil pipe.

3. Is to be Eastern Expanded Metal Co.'s B No. 27 gage, wire lath, or equivalent.

Plastering Partitions:

1. All surfaces covered with wire lathing and each side of each partition are to be given two coats of plaster as follows:

(a) The scratch coat is to be made of 1 part of Portland cement to 4 parts of lime putty, the lime putty being made of 1 part of best lump lime thoroughly mixed with about 3 parts of sand and best long cattle hair, in as large a quantity as is practicable.

(b) The finish coat is to be made of 1 part of Portland cement to 3 parts of sand with only enough best lump lime to make it work properly under the trowel.

(c) The scratch coat is to be thoroughly dry before the finish coat is applied.

(d) The cement for the finish coat is to be mixed in immediately before the mortar is to be used, and no mortar which has set or which is left from the previous day's work is to be used.

(e) The finish coat is to be "floated."

2. The thickness of partitions on all floors, as marked on the drawings, indicates the over-all thickness plaster to plaster; 2-in. partitions are to have no air spaces. Wider partitions are to have at least $\frac{3}{4}$ in. of plaster on each side, the remainder being air space.

3. The necessary openings for pipes, conduits, wires, sleeves, carriers, etc., are to be made, and all cutting, patching, and repairing of plaster work are to be included.

Plastering of First Floor Ceilings:

1. The entire ceilings of the hall, including the entrance vestibule, the reception room, entry, toilet room, and the private office are to be plastered as follows: A brown coat of Windsor cement with a finish coat composed of 1 part of plaster of Paris and 2 parts of lime putty, trowelled to a smooth finish.

Lathing and Plastering of Exterior Walls and Piers:

1. All of the exterior piers and girders in the rooms whose ceilings are to be plastered as specified above, are to be similarly plastered, where exposed.

2. All curtain walls in the rooms whose ceilings are to be plastered are to be metal furred, wire lathed and plastered like the interior lath and plaster partitions.

Cornice Work:

1. (a) Is to be made to detail of equal parts of lime, putty, and plaster of Paris. All furring required is to be done in metal.

(b) Is to occur in those places for which plastered ceilings are specified, except in toilet rooms and entry out of private office, and as indicated on drawings.

Exterior Plastered Walls:

1. The curtain walls on the east side above the level of the first floor and the walls of the elevator shaft above the roof are to be built up of metal studs with metal lath. The material for these walls is to be of size and arrangement, as for 4-in. partitions before specified.

2. These walls are to be plastered inside and outside with cement plaster as before specified, two coat work, the outside being left with a little rougher finish than the inside, which will be like inside partitions.

CARPENTRY

Forms and Molds:

1. Are specified for the concrete construction under the heading "Concrete Work." The specifications under "Carpentry" do not refer to forms and molds.

Lumber:

1. All wood is to be free from large or loose knots, and commercially free from sap and other defects.

2. All wood, dimensions, quality, etc., are specified under the various headings below.

Windows:

1. Are to be provided and set where indicated, in all walls and partitions, being double hung, swing, or fixed sash, as marked.

2. Are to be of the dimensions and designs indicated, and in accordance with detail drawings.

3. Are to be built of thoroughly seasoned, kiln dried, white pine, including frames and sash, where exterior, but of cypress in interior partitions.

4. Fixed and swing windows are to have rebated plank frames of dimensions indicated.

5. Double hung windows are to have (a) full box frames, (b) hard-pine pulley stiles with removable sections, (c) cast iron weights, balancing the sash so that they will run easily, (d) No. 8 Samson Spot Cord hanging cords, (e) bronze weight pulleys, 2¼ in. in diameter, with bronze finished face plates, (f) molded muntins and lipped and coped meeting rails.

6. All windows are to be as tight-fitting as is consistent with easy operation.

7. All window frames are to be thoroughly primed before leaving shop. (See "Painting and Glazing.")

8. (a) The staff beads are to be removed for and replaced after pointing; (b) the pointing is specified under "Cleaning and Pointing" above.

9. All windows are to be equipped with the hardware specified and included in the "Hardware Allowance."

10. All double hung windows are to have stop blocks to prevent lower sash from being raised too far.

11. Metal sash and frames are specified under "Roofing and Sheet Metal Work."

Transoms:

1. Are to be located over doors where so indicated on the drawings.

2. Are to be hinged at the bottom and made like swinging windows.

Doors:

In General:

1. Are to be provided and hung where indicated, being swing, sliding, glazed, etc., as marked.

2. Are to be of the dimensions and designs indicated and in accordance with detail drawings.

3. Doors which are not to be finished natural are to be thoroughly primed before leaving the shop. (See "Painting and Glazing.")

4. (a) Are to be equipped with the hardware included under "Hardware Allowance"; (b) tinned doors are to be equipped with hardware, exclusive of the "Hardware Allowance."

Tinned Doors:

1. Are to be installed where so indicated.

2. Are to be of Victor Mfg. Co. make, Newburyport, Mass., or Coburn Mfg. Co., Holyoke, Mass., in accordance with the latest rules of the National Board of Fire Underwriters, and bearing the "Underwriter's Label."

3. Are to be not less than 2¼ in. thick before tin is applied.

4. (a) Sliding tinned doors are to be hung on steel tracks, with brackets, roller hangers, binders, rope pulleys, bumpers, stay rolls, chafe clips, fusible links, weights, pulls, etc., complete.

(b) Attachment to doors and walls is to be made by through bolts.

5. (a) Swinging tinned doors are to be similarly equipped complete, except those at basement and first floor levels leading to stairs, which

will have fusible links and weights, only the hardware being in the "Hardware Allowance," including strap hinges three-quarters across the doors, eye-bolts, latches operated from sides, etc.

(b) Are to be glazed where indicated with wired glass $\frac{1}{4}$ in. thick.

6. Frames for swinging tinned doors are specified under "Steel and Iron Work."

7. All of the above hardware, except for doors to stairs at basement and first floor levels is exclusive of that included in the "Hardware Allowance."

Exterior Doors:

1. Where not tinned are to be made of thoroughly kiln dried white pine, panelled and glazed, as indicated.

2. Are to have rebated plank frames of kiln dried white pine.

Interior Doors:

1. Where not tinned, are to be of cypress, except that doors opening into reception room, entry between reception room and private office, toilet room adjacent, and private office are to be quartered oak on sides opening into these rooms.

2. Are to have strong rough stud frames securely fastened in place.

3. Are to have double rebated, finished frames, covering the rough frames completely. (See also "Standing Finish.")

4. Are to be of dimensions indicated, with cross panels, as shown.

Water Closet Doors:

1. Doors between toilet rooms and water closets and between coat room on first floor and the adjacent water closet, are to be made of $\frac{7}{8}$-in. cypress, with stiles and rails, with simple panels.

2. Are to be hung about 1 ft. above the floor.

Rolling Steel Doors:

1. Are specified under "Steel and Iron Work."

Builders' Hardware:

In General:

1. Is to be provided and installed complete, as may be necessary for the proper finishing of the work, including all nails, screws, weights, chains, cords, pulleys, etc., and all the hardware covered by the hardware allowance specified below.

2. Is to be of the best quality and of proper size, weight, and design, for the work, in accordance with the best factory practice.

Hardware Allowance:

1. The contractor is to make an allowance of four hundred and fifty dollars ($450.00) for locks, butts, hinges, kick and push plates, base knobs, catches, checks, springs, lifts, sockets, hooks, stop bead screws, etc., for windows and doors, in addition to such of these articles as are specified above for tinned doors.

2. This allowance is to include the delivery of the articles at the building, but is not to include care of them after delivery or installation.

3. The contractor is to receive the articles covered by the allowance and is to install them complete.

Standing Finish:

1. Except otherwise specified, windows in interior walls or partitions are to have plain trim, $\frac{7}{8}$ in. thick and 5 in. wide, with plain corner blocks, 1 in. thick, and simple molded stools and rebated aprons, with rounded edges, nosings and simple undermoldings, essentially as indicated, the finish being installed on both sides.

2. (a) Window and door finish on exterior is to be as indicated on the drawings and as detailed.

(b) The windows and doors in private office, toilet room, entry, reception room, vestibule and hall are to have $\frac{7}{8}$-in. molded and mitred architraves, with $\frac{7}{8}$-in. finish pieces on the mullions and at the jambs; the above finish together with the stools and aprons being molded to detail essentially as indicated.

(c) Trim for windows and door to stairs, first floor, is to be as speci·fied under "Roofing and Sheet Metal Work."

3. Except as otherwise specified, frames for interior swing, untinned doors are to have plain trim, $\frac{7}{8}$-in. thick and 5 in. wide, with plain corner blocks 1 in. thick, and plinth blocks.

4. Water closet door frames are to have simple moldings to match the moldings on the water closet partitions.

5. Rail and double swing gate of cypress is to be built to detail for opening between hall and general office. Rail is to have $1\frac{1}{8}$-in. molded quartered oak shelf, on top as shown.

6. Molded base-boards, with removable grooved piece for telephone wires, chair rails, and picture moldings are to be installed in reception room, private office, entry, and toilet on the first floor.

7. The molded base-board with dado of matched and grooved $\frac{7}{8}$-in. cypress sheathing 3 in. wide and dado cap, all as detailed, is to be installed in vestibule and hall.

8. Wood panelled screen with doors and side lights and simple molded cornice, all of cypress, to detail, is to be built in hall as indicated.

9. All necessary nailing strips, blocks, etc., are to be provided in time for building into the concrete and plaster partitions.

10. All interior finish except as otherwise specified is to be cypress, dressed ready for painter's finish.

11. Finish in reception room, entry between reception room and Private office, in private office, in toilet room adjacent, is to be quartered oak.

Shelves and Hook Strips:

1. (a) Are to be installed as indicated.

(b) Hook strips are to be $\frac{7}{8}$-in. cypress.

(c) Shelves are to be installed with cleats, etc., of $\frac{7}{8}$-in. cypress of width and length indicated.

Water Closet Partitions:

1. Are to be of cypress, matched sheathing, planed on both sides, with door frames, top and bottom rails, beading, etc., as indicated on the drawings.

2. Are to be supported on iron legs provided as specified under

"Plumbing" below; all necessary braces for making the partitions rigid are to be installed.

Flag Pole:

1. There is to be a flag pole made of Nova Scotia spruce, with 6-in. gilded ball on top and of height and dimensions indicated on drawings.

2. It is to be equipped with halyards.

Backboard:

1. Are to be provided as may be necessary for plumbing equipment.

Shelves for Gas and Water Meters:

1. Are to be provided and erected with brackets as directed at the building.

Trench Cover:

1. The trench cover in basement floor is to be of hard pine plank, 2 in. thick, planed on top and edges and nicely fitted.

Wooden Grating:

1. There is to be a wooden grating over scupper in basement near cleaning room, as indicated.

2. The grating is to be made of pine and is to be strong and rigid and so set as to be flush with the floor.

3. The grating is to be removable and is to be made with 1-in. by 2-in. stock set on edge, 3 in. on centers, running both ways aross the grating.

Stair Rails:

1. Oak hand rails, of design approved by the Engineers, are to be provided and attached to stair rails from first floor to third floor.

STEEL AND IRON WORK

In General:

1. The steel for reinforced concrete is specified under the heading "Reinforced Concrete." The specifications under the heading "Steel and Iron Work" do not refer to reinforced steel for the concrete.

Materials:

1. Cast iron is to be free from blow holes, flaws, or other imperfections, and is to be cast smooth and true.

2. Steel shapes are to be in accordance with the standard of the Association of American Steel Manufacturers.

3. All connections are to be made so as to develop the full strength of the pieces connected.

4. Shop drawings are to be submitted to the Engineers for approval before construction is begun.

Painting:

1. All steel and iron work is to be thoroughly cleaned and given one coat of Dixon's or Superior graphite paint before leaving the shop.

2. All parts which are inaccessible after erection are to be given a second coat immediately before erection; all parts accessible after erection are to be given a second coat after being put in place.

3. This painting is in addition to any painting specified under the heading "Painting and Glazing."

Anchors, Bolts, Plates, Etc.:

1. Are to be provided as required for all parts of the work.

2. All plates, knees, etc., are to be drilled and countersunk for screws as may be necessary.

Steel and Iron Work in Lath and Plaster Partitions:

1. All metal studding, etc., is specified under "Lathing and Plastering."

2. The iron door frames are specified below.

Door Frames for Tinned Doors:

1. The door frames for tinned doors, except in plaster partitions, are to be made of heavy angle irons, carried over tops as well as up the sides and, for swing doors, provided with suitable rebate angles attached to top and sides, and all securely fastened in place.

2. The door frames for tinned doors in plaster partitions are to be made of heavy channel irons, the full width of partitions, top and sides, and, for swing doors, provided with rebate bars.

3. The erection of these frames is specified under "Lathing and Plastering."

4. All frames are to be drilled and tapped for all hardware for the doors, including (a) heavy angle iron supports on each side of partition, drilled and made ready for sliding door hangers; (b) provision for attachment of chafe clips, stops, etc., etc. (For specifications for the doors and their hardware, see "Carpentry.")

Frames for Metal Sash:

1. Frames of windows in stair-well walls above first floor are to be of channel irons, top, bottom, and sides, with rebate strips all around on each side of sash, all essentially as detailed.

Rolling Doors:

1. "Kinnear" rolling steel doors are to be installed where so indicated, with all supports, guides, raising and lowering devices, locking devices, etc., complete.

2. Are to have fuse links and are to be made to close automatically in case of fire.

3. Door frames with all supports and attachments are to be included.

Iron Stairs:

1. Are to be installed from first-floor level to roof level as indicated.

2. Are to be strongly supported from basement floor by steel struts in rear corners as necessary and on concrete landings at floor levels.

3. Stairs from first-floor level to third-floor level are to be made with

(a) Heavy channel-iron panelled stringers, closed string, checkered cast-iron treads and landings and plain risers, making a closed stair.

(b) Treads, landings, and risers strongly supported between stringers, with reinforcement of approved shape secured to same.

(c) Treads and landings to have nosing and under molding.

(d) Are to have simple moulded piece covering joints where plaster walls meet inside stringer.

(e) Are to be built to carry a live load of 100 lb. per square foot in accordance with detail drawings submitted by the Contractor for approval by the Engineers.

4. Stairs from third floor to roof are to have:

(a) heavy channel-iron stringers, checkered cast-iron treads;

(b) Checkered cast-iron landing; all strongly supported and framed together.

Railings:

1. Are to be installed complete for stairs and landings from basement to roof level and for landing in roof house at roof level.

2. The rail for basement stairs is to be on one side only and is to be a single 1½-in. pipe, securely fastened by supports bolted through partition with flange on back.

3. The hand rails on the stairs from first floor to third floor are to be ornamental wrought iron and cast iron, with posts, balustrades, etc.

4. All other rails are to be "double rail" of 1½-in. pipe, with 1½-in. pipe stanchions, about 6 ft. on centers.

5. All railings are to be strongly and rigidly supported, with flanges at all wall and floor contacts, sockets being built in where necessary.

6. All pipe and fittings are to be standard rail pipe and fittings.

Safety Treads:

1. Mason safety treads are to be installed on all treads, including edges of landings, of all stairways and the front entrance steps, excepting stairs from third floor to roof.

2. (a) Those on concrete steps and landings are to be set flush and securely attached to anchors embedded in the concrete and are to be 3½ in. wide with nosing.

(b) Those on iron stairs are to be 4 in. wide without nosing.

(c) All are to come within 4 in. of each end of tread.

Pressed Steel Thresholds:

1. Are to be provided and securely anchored in place where so indicated on the drawings.

2. Those at elevator doors are to overhang the well as shown and are to be provided with faces slanting back to edge of slab or girder, these faces to be not less than 8 in. deep, all essentially as indicated.

Steel Framing:

1. Is to be provided as indicated for the heavy framing of roof house, in addition to the steel in plaster wall previously specified.

2. Is to be provided and securely erected outside of elevator well for attaching elevator guides.

Hanger Sockets:

1. Are to be provided for building into concrete.

2. Are to be of socket type, drilled and tapped for ¾-in. bolts, and of such design as to insure proper support for hanging shafting, overhead tracks, etc.

3. Are to be made so that they will finish flush with the beams, girders or slabs in which they are installed.

4. Samples are to be submitted to the Engineers for approval before selection.

Flag-pole Supports:

1. Heavy galvanized iron flag-pole supports and galvanized iron halyard belaying pin are to be provided and installed complete.

Protection Strips:

1. Angle iron protection strips are to be provided and installed complete at rear basement doorways as indicated.

Thimble and Cleanout Door:

1. Are to be provided for installation in chimney, the thimble being for 14-in. smoke pipe, the cleanout door, which is to be provided with heavy iron frame, being 12 in. by 14 in.

ROOFING AND SHEET-METAL WORK

Materials:

Tarred Felt:

1. Is to be "Barrett Specification Felt."
2. Is to weigh not less than 14 lb. per 100 sq. ft. of single thickness.

Pitch:

1. Is to be "Barrett Specification Coal-tar Pitch."

Gravel:

1. Is to be of such size as to be rejected by a ¼-in. mesh screen and to pass through a ⅝-in. mesh screen.
2. Is to be clean and dry, being heated immediately before use if the weather is cold.

Roofing:

1. The entire roof of the building is to be 5-ply, tar and gravel, applied in accordance with the best practice, essentially as follows:

(a) The concrete is to be covered with pitch mopped on uniformly, special care being taken that the pitch is very hot and very thoroughly mopped on.

(b) Two layers of tarred felt are to be laid, each sheet lapping the preceding 17 in. and being mopped back with pitch the full width of each lap.

(c) Pitch is to be uniformly mopped on, covering the whole surface.

(d) Three layers of tarred felt are to be laid, each sheet lapping the preceding 22 in.; each layer is to be mopped back with pitch the full width of the 22 in. under each lap.

(d) The entire surface is to be uniformly spread with pitch not less than $\frac{3}{16}$ in. thick, with gravel spread into it, while the pitch is still hot.

(f) Special care is to be taken to cover and make tight around all supports on the roof and to finish properly around all plumbing pipes, vapor pipes, and similar apparatus extending through the roof.

Flashing and Counter-flashing:

1. Is to be done with 16-oz. copper, where indicated on the drawings and where necessary for the proper completion of the work, including (a) on parapet walls on north and west elevations, where the counter-flashing is to be carefully cemented into reglets; (b) on other parapet walls where the flashing is to be carried up and over and nicely dressed down, as indicated; (c) around all vapor and soil pipes extending through the roof, in which cases each pipe is to have expansion sleeve soldered on and is to be counter-flashed with inverted copper cone attached to

pipe; (d) around the roof house the flashing or counter-flashing is to be carried well up under the plaster of the walls.

2. All flashing is to be carried under the roofing at least 2½ in. and up high enough on walls, skylights, etc., to give lap between flashing and counter-flashing of at least 4 in. and more where so indicated on the drawings.

Leaders:

1. Roof drains, with 18-oz. copper boxes and copper-wire screens are to be installed, where indicated.

2. Four-inch copper leads are to be run from the roof boxes to the conductors with brass sleeves soldered on, ready for calking into the conductors, the calking being done as specified under "Plumbing."

Guarantee:

1. The Contractor is to guarantee all parts of the roofs, skylights, flashing, roof drains, etc., to be tight for a period of five years from the date of completion of this contract.

2. Any leaks or other defects due to improper workmanship or materials discovered within five years are to be made good by the contractor free of charge.

Door and Sidelights in First Story Leading to Stairway:

1. Are to be of metal together with the trim on both sides of the partition.

2. The stiles and rails of doors are to be tinned as specified under "Carpentry." The glazed portion is to be fitted with hollow galvanized-iron sash divided into lights as shown.

3. The sidelights are to be hollow galvanized-iron sash divided into lights as shown.

4. The trim is to be of galvanized iron molded to detail.

Interior Metal Sash:

1. In addition to that above specified is to be installed where indicated of hollow galvanized iron divided into lights as shown; the frames for these windows are specified under "Steel and Iron Work."

Exterior Metal Sash:

1. Is to be installed where indicated of hollow galvanized iron divided into lights as shown.

Glazing of Metal Sash:

1. Is to be done with wired glass ¼ in. thick.

Copper Ventilators:

1. Are to be installed where indicated of the sizes shown.

2. Are to be equipped with damper, controlling chains, etc., etc.

ELEVATOR

Drawings:

1. The drawings indicate general dimensions only, and all dimensions are to be verified at the building.

2. The Contractor is to furnish all detail drawings required.

Number, Type and Make:

1. There is to be one, direct-connected, alternating current, electric freight elevator.

2. It is to be of[1] . . . . make.

Travel, Speed and Load:

1. The travel is to be from the basement floor level to the third-floor level, about 32 ft., 6 in., the exact travel being determined at the building.

2. The speed is to be 55 ft. per minute.

3. The capacity at the above speed is to be about 3000 lb. exclusive of the car, but slight variations will be allowed to fit stock sizes, provided that said variations are approved by the Engineers before contract is signed.

Car:

1. The car is to be a standard freight platform, with heavy angle-iron frame and hardwood floor, thoroughly braced.

2. The car is to be as large as the well size will permit, approximately 5 ft. by 6 ft.

3. The side of the car adjacent to the weight guides is to be sheathed 5 ft. high and strongly braced.

Machine and Motor:

1. Are to be installed at top of elevator well.

2. Are to be supported on and bolted to I-beams provided and installed by the Contractor, complete, with all bearing plates, holding down bolts, anchors, etc., required.

3. Are to be of the latest and best pattern of the make selected.

4. The motor is to be of alternating-current type, arranged for 550 volts, three-phase, 60-cycle.

5. The bearings are to be self-oiling.

Sheaves and Sheave Beams:

1. The sheaves are to be of cast iron, accurately turned and grooved, mounted on steel shafting turning in Babbitt boxes.

2. The sheaves are to be supported on steel beams provided and put in place by the Contractor complete, with all bearing plates, holding down bolts, anchors, etc., required.

3. All drilling, cutting, and patching in connection with setting of beams will be done as specified under "Concrete Work" upon demand of the sub-contractor for the elevator, but any cutting done by the sub-contractor for the elevator must be patched by him also.

Counterweights:

1. There are to be two counterweights, one for counterbalancing from the car itself, and one for counterbalancing from the drum.

2. Both counterweights are to be of cast iron.

Guides:

1. Are to be of maple on hard pine posts, for both the car and the counterweights.

2. Are to be strongly supported, all steel and iron work other than that specified above, under "Steel and Iron Work," and all drilling and patching being included.

Cables:

1. Are to be best wire-hoisting cables of proper sizes.

[1] Not to be filled in until the contract is awarded.

Method of Control:

1. The car is to be controlled by a steel wire shipping rope with ball stop at top and bottom of run, and with suitable lock.

Safety Devices:

1. The elevator is to be provided with (a) safety device for locking the car if it descends too fast; (b) slack safety device; (c) automatic stopping device for stopping the car at top and bottom of its travel.

2. The elevator is to be equipped with safety brake, which is automatically set when the current is cut off by attendant or by operation of one of the safety devices.

Gates:

1. Each opening, of which there are seven, is to be equipped with semi-automatic gate.

2. Each gate is to be double rail, strong and substantial, and is to operate smoothly and surely without binding or catching.

3. Each gate is to be hung from both ends, and is to be rigid and so attached to its guides that it cannot be sprung from them.

4. Counterweights are to have neat finished wood weight-boxes.

Electric Lights and Wiring:

1. (a) The elevator is to be provided with an electric-light fixture with heavy wire protecting guard, switch and wiring, and cable to outlet box at point half way up the well.

(b) The outlet box in the well and the wiring as far as this box, will be provided under another contract.

2. The Contractor is to provide lugs (of the proper size for receiving the service wires) on his controller to which electrical connection is to be made under another contract.

Grafting:

1. The contractor is to provide and erect complete over the elevator well an iron grating made of 1½- by ½ in. bars, set on edge, 1½ in. on centers, with separate bolts not more than 2 ft. apart, and with heavy channel-iron frames.

2. In addition, the Contractor is to provide and erect a ½-in. mesh galvanized-iron netting to prevent the possibility of tools or other objects falling through the grating and down the well.

Painting:

1. All structural steel and iron work, except the bearing surface of guides, etc., is to be given a priming coat before erection and three coats of lead and oil paint after erection, of color selected by the owner.

Starting up and Testing:

1. The contractor is to start up and put the elevator in proper operation.

2. The contractor is to test the elevator for capacity, speed, proper operation, etc., complete, in the presence of the Owner and the Engineers.

3. A suitable weighed load for testing will be provided by the Owner, but the Contractor is to load and unload the elevator car as may be required.

Guarantee:

1. The Contractor is to guarantee the elevator to be free from de-

fects in workmanship, material, or design for the service required, and is to guarantee to make good, free of charge, any such defects which may come to light within one year from the completion of this contract.

Laws:

1. The Contractor is to fulfill all the laws, ordinances, and requirements of state, city, insurance companies, etc.

Time of Doing Work:

1. All necessary measurements are to be taken at the building at the earliest possible moment, to which end the contractor for the building and the sub-contractor for the elevator are to co-operate.

2. The elevator installation is then to be pushed to completion as rapidly as possible, so that it may be used by the Owner even before the contract for the building is entirely completed.

PAINTING AND GLAZING

Painting:

Preparing Work and Puttying:

1. All surfaces to be painted are to be properly cleaned and prepared for painting, the rust being removed from metal, and all surfaces being made smooth, the woodwork being sand-papered and all nail holes being properly puttied, the putty being colored with paint of the same color as the finish coat is to be.

Lead and *Oil Paint:*

1. Is to be made of pure new lead and best American linseed oil.

Colors:

1. Are to be subject to the approval of the Engineers, different shades being used on walls above and below chair rails, etc.

2. Samples of the various colors and shades are to be submitted to the Engineers for approval before the painting is done, and all colors and shades finally used are to be in accordance with the approved samples.

Priming:

1. The priming of steel and iron work is specified under the heading "Steel and Iron Work."

2. All window and door frames are to be thoroughly primed in the shop before being shipped to the building.

3. All exterior woodwork to be painted is to be given one priming coat before or immediately after erection, special care being taken that all surfaces adjacent to or set in masonry are primed before erection.

Exterior Painting:

1. Three coats of lead and oil paint, in addition to the priming coat, are to be applied to (a) all exposed exterior woodwork, including doors and frames, window sash and frames, flag-pole, etc.; (c) all metal-covered window and door frames, sash, etc.

Interior Painting:

1. Three coats of lead and oil paint are to be applied to (a) all exposed interior steel and iron work, including iron stairs and ladders and supports, steel framing, railings, elevator sheave beams and plates,

tinned doors, frames, gratings over elevator wells, elevator machines and motor, rolling steel doors and supports, casings, pipe sleeves and plates, not N. P.; (b) all exposed interior woodwork, except hardwood, including windows, frames, sash, pine plumbing tanks, backboards, etc.; (c) all metal and metal-covered window and door frames, sash, etc.; (d) all supports for w. c. partitions and for wooden sinks· all iron sinks, angles on wooden sinks.

2. All buried plumbing piping is to be given one coat of black asphaltum.

3. (a) All concrete and plaster walls, columns, piers, partitions, etc., in the vestibule, hall, private offices, reception room, entry, and office toilet room on the first floor are to be given one coat of a cement coating approved by the Engineers, followed by one coat of the corresponding enamel or gloss coating, different colors being used, and above the chair rail, where there is one, and all colors being approved by the Engineers from samples submitted.

(b) The ceilings of the above rooms are to be tinted one coat.

4. (a) All other concrete and plaster walls, columns, piers, partitions, etc., except in the basement, are to be given one coat of approved cement coating, followed by one coat of corresponding enamel or gloss coating, up to a height 5 ft. 6 in. from the floor level, and one coat of cold water paint, sprayed on, above.

(b) All other concrete and plaster ceilings, girders, beams, etc., except in the basement, are to be given one coat of cold water paint, sprayed on.

5. (a) The walls, columns, piers, and partitions in the cleaning room in the basement are to be painted above the tiles, as specified for the walls in the offices.

(b) The ceiling girders, etc., in the cleaning room are to be given one coat of cold water paint.

Bronzing:

1. All exposed water, soil, vent, rain water and gas piping, except brass piping, is to be given two coats of bronze, different colors being used for different kinds of piping, as directed by the Engineers.

Lettering:

1. Each toilet room and locker room is to be nicely lettered, with letters 1½ in. high, indicating name and use of room.

2. Each door opening on to elevator well is to be marked with letters 2½ in. high "ELEVATOR."

3. All wording is to be approved by the owner.

4. All lettering is to be done by experienced sign painters, with type of paint and color best suited to the requirements in each case.

Finish:

1. One coat of oil followed by one coat of spar varnish is to be applied to (a) all sheathing partitions and water closet doors, (b) all hardwood trim and finish, except where otherwise specified below, (c) hardwood doors, (d) elevator gates.

2. One coat of oil is to be applied to all pulley stiles.

3. All hardwood finish, doors, etc., in vestibule, hall, private office,

reception room, entry and office toilet room, and the railing and shelf and gate at office entrance are to be finished with one coat of stain followed by one coat of shellac rubbed down, followed by one coat of shellac and wax rubbed down.

Glazing:

1. Plate glass is to be used for glazing the front entrance transom and doors, and doors and windows in "screen" between vestibule and hall.

2. Best quality factory ribbed glass is to be used for (a) all interior glazing except certain individual lights which are to be clear glass, such as between private office and main office on first floor, etc., the exact lights being selected at the building and except where wired glass is indicated.

(b) All lower sash in windows in toilet rooms and wash rooms.

3. Wired glass for interior glazing and for exterior glazing is included under "Carpentry" and under "Roofing and Sheet Metal Work."

4. All other glazing is to be done with double thick, first quality, American glass.

5. All glazing is to be repaired at the completion of the building.

SECTION 2

CONSTRUCTION

CHAPTER XVIII

MATERIALS

78. Cement.—Portland cement only should be used in reinforced-concrete construction. Natural and slag cements are available for construction purposes, but are unsuitable for use except in unexposed structures where mass and weight rather than strength are essential features. In reinforced work proper or in structures exposed to the action of the elements, Portland cement should invariably be employed because of its uniform quality and the high strength which it rapidly acquires.

Portland cement is made by artificially mixing lime and clay in proper proportions, heating the mixture to the point of fusion, and then finely pulverizing the clinker. The materials from which Portland cement is manufactured vary with the locality, but Portland cement can be made anywhere by burning and grinding a proper mixture of carbonate of lime and a suitable clay.

The basic elements of Portland cement are silica, alumina, and lime. Ingredients such as iron, magnesia, alkalies, sulphuric acid, carbonic acid, and water also occur in varying quantities replacing some of the basic elements. The constituents should approximate the following limits:

	Per cent.
Silica	20 to 24
Alumina	5 to 9
Iron oxide	2 to 4
Lime	60 to 63.5
Magnesia	1 to 2
Sulphur trioxide	1.5

The essentials of Portland cement—namely: silica, alumina, and lime—are obtained from six different sources.

(1) Cement rock and limestone.
(2) Limestone and clay.
(3) Marl and clay.
(4) Chalk and clay.
(5) Blast-furnace slag and limestone.
(6) Alkali waste and clay.

Cement rock (argillaceous limestone, low in magnesia) and limestone are chiefly used in the Lehigh district and constitute the raw materials used for about two-thirds of the Portland cement manufactured in the United States.

In order to insure a satisfactory and uniform grade of cement, shipments should be tested according to rules laid down by the American Society of Civil Engineers and the results should conform to the requirements of the standard specifications prepared by the American Society for Testing Materials (Appendix, Section H). Copies of these publications may be obtained from the societies named.

Results of tests depend largely upon the method of testing and hence it is important that definite and uniform methods be employed. The kinds of tests usually made are as follows: (1) specific gravity; (2) fineness; (3) time of setting; (4) tensile strength of neat cement and of sand mortar; (5) soundness, or constancy of volume. A chemical analysis is also desirable on large work. The tests for soundness and strength are called primary tests, while the other tests are called secondary since they give only additional information which is of little value in itself.

Cement testing is a difficult process and should be entrusted only to experienced, well-qualified men. It is a well-known fact that the personal factor has considerable to do with the results obtained in cement testing and, on this account, there is a vast amount of dissatisfaction with the methods specified at the present time. Moreover, results by utterly untrained or careless operators are really worse than nothing and may be positively misleading. The comparative results by any one experienced observer, however, are generally consistent and are of value. From the above it should be recognized that, if cement tests are made, it is worth while to have them made well, even at a possible increase of expense.

The cement used in testing should be representative. Generally 1 bag in every 40 is sampled, or, in case of delivery in bar-

rels, 1 barrel out of every 10. These small samples are usually mixed together to make an average sample. The whole is then taken to the testing laboratory, and there submitted to the standard tests. Usually 8 or 10 lb. of cement is taken from each car load for testing purposes. An average sample of a package may be obtained by means of a sampling auger (such as is used by butter or sugar inspectors) inserting it from top to center in bags, and from side to center in barrels. When sampling from barrels, a hole is made for the purpose in the center of one of the barrel staves midway between the heads.

Adulterations in Portland cement have generally been looked upon as detrimental to its quality, and the cement specifications in common use are of a character to exclude any grinding in of cheap material after calcination. Recent investigations have shown, however, that some forms of adulteration are possible without injury to the cement and in many cases may produce a decided improvement in every desired quality. The question at once arises—why should specifications for Portland cement be changed so as to admit of such a practice, thus materially reducing the cost? The principal reason lies in the fact that with what is known at the present time, specifications permitting adulteration would be difficult to enforce and the results obtained with adulterated material would be uncertain. The specific gravity test of cement is mainly for the purpose of detecting adulteration such as clay, slaked lime, sand, ashes, natural cement, or products of natural rock-like ragstone or tufa. The specific gravity test, however, should be taken only as indicative and should be verified by other tests before rejecting a material which does not come up to standard. Chemical analysis serves as a valuable means of detecting adulteration, and also shows whether those elements believed to be harmful are present in too large quantities.

It is well known that fine grinding has a very great influence on the properties of the cement. The finer the grinding, the better the quality, since fine particles will more easily cover the sand grains, making mortar much stronger and allowing the use of a larger percentage of sand. A Portland cement of good quality should be fine enough to pass at least 92 per cent by weight through a No. 100 sieve and 75 per cent through a No. 200 sieve.

The time of setting of a cement may vary within wide limits and is no criterion of the quality, but it is important in the fact

that it indicates whether or not it can be used advantageously in ordinary construction. A cement may set so quickly that it is worthless for use as a building material (since handling cement after it commences to set weakens it and causes it to disintegrate), or it may set so slowly that it will greatly delay the progress of the work. Long-standing cements absorb moisture from the air and lose their hydraulic property. A moderate amount of *seasoning* in weather-tight sheds, however, is often helpful to secure good results. Fresh cement contains small amounts of free or loosely combined lime which does not slake freely like ordinary lime and causes expansion, endangering the structure in which it is used in this condition. During the time of seasoning the free lime is changed to carbonate of lime and in this state the cement does not swell. Usually cement is seasoned at the mills before shipping, but, with the best mills, the stock house may run so low in periods of rush that a chance will be taken on fresh material. Well-seasoned cement is generally lumpy, but the lumps are easily broken up. If, however, the cement has been subjected to excessive dampness, or has been wet, lumps will be formed which are hard and difficult to crush. Aside from the consideration of age, the conditions which accelerate the setting are: finely ground and lightly burned material, dry atmosphere, small amount of water used in gaging, and high temperature of both water and air.

There are two distinct stages in setting: (1) the initial set; and (2) the hard set. The best cements should be slow in taking the initial set but after that should harden rapidly. Portland cement should acquire the initial set in not less than 30 minutes, and hard set in not less than one hour nor more than 10 hours.

The object of testing cement in tension is to obtain some measure of the strength of the material in actual construction. In other words, tests of tensile strength are made primarily to determine whether the ingredients of the cement and the process of its manufacture are such that a continued and uniform hardening may be expected in the work, and whether it will have such strength when placed in mortar or concrete that it can be depended upon to withstand the strain placed upon it.

The small shapes made for testing are called *briquettes* and have a minimum cross-sectional area of 1 sq. in., that is, at the place where they will break when tested. Tests are made with bri-

quettes of neat cement and also with briquettes made of cement mortar (one part cement, three parts standard sand from Ottawa, Illinois). The neat cement tests measure directly its quality while the mortar tests are more nearly a true measure of its behavior under actual conditions.

It is customary to store the briquettes, immediately after making, in a damp atmosphere for 24 hours. They are then immersed in water until they are tested. This is done to secure uniformity of setting, and to prevent the drying out too quickly of the cement, thereby preventing shrinkage cracks which greatly reduce the strength.

Specifications for tensile strength of cement usually stipulate that the material must pass a minimum strength requirement at 7 and 28 days. This is required in order to determine the gain in strength between different dates of testing so that some idea may be obtained of the ultimate strength which the cement will obtain. A first-class cement, when tested, should give the values for tensile strength stated in the specifications of the American Society for Testing Materials.

W. Purves Taylor in "Practical Cement Testing" gives the following rules for accepting or rejecting material on the results of tensile tests:

"At 7 days: Reject on decidedly low sand strength. Hold for 28 days on low or excessively high neat strength, or a sand strength barely failing to pass requirements.

"At 28 days: Reject on failure in either neat or sand strengths. Reject on retrogression in sand strength, even if passing the 28-day requirements.

"Reject on retrogression in neat strength, if there is any other indication of poor quality, or if the 7-day test is low; otherwise accept.

"Accept if failing slightly in either neat or sand at 7 days and passing at 28 days."

It must be remembered that the sand test is the true criterion of strength, and no cement failing to pass this test should be accepted, even though the neat tests are satisfactory.

A cement to be of value must be perfectly sound; that is, it must remain constant in volume and not swell, disintegrate, or crumble. Excess of free or loosely combined lime which has not become sufficiently hydrated, excess of magnesia and alkalies, and coarse grinding, are all causes of unsoundness. Tests for soundness are among the most important to be made upon cements and should extend over considerable time to fully develop

possible inherent defects. The usual way is to form small pats of neat cement about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge. These pats should remain a definite time: (1) in air; (2) in water; and (3) in an atmosphere of steam above boiling water. To pass the soundness tests satisfactorily, the pats should remain firm and hard, and show no signs of cracking, distortion, or disintegration. The steam tests are what are called *accelerated* tests and are for the purpose of developing in a short tine (five hours) those qualities which tend to destroy the strength and durability of a cement. In the present state of our knowledge it cannot be said that cement should necessarily be condemned simply for failure to pass the accelerated tests; nor can a cement be considered entirely satisfactory simply because it has passed these tests. The air and water tests are called *normal* tests and extend over a period of 28 days.

Full specifications similar to those of the American Society for Testing Materials are advisable for important work whether large or small. When purchasing by such specifications it may often be unnecessary actually to test the cement except for set, soundness and fineness, but the full specifications are recommended so that, if the cement does not work satisfactorily, it may be more carefully examined and unused portions rejected. Some large purchasers have apparatus for making complete cement tests in the field, but it is customary to have the tests made at some testing laboratory. In unimportant construction it is usually safe to use a first-class American Portland cement without testing.

Cement should be stored within a tight, weather-proof building, at least 8 in. away from the ground and an equal distance from any wall, so that free circulation of air may be obtained. In case the floor of the building is laid directly above the ground, it would be well to give the cement an additional 8-in. elevation by means of a false floor, so as to insure ventilation underneath. The cement should be stored in such a manner as to permit easy access for proper inspection and identification of each shipment. A period of at least 12 days should be allowed by the contractor for the inspection and necessary tests.

Where cement in bags is stored in high piles for long periods, there is often a slight tendency in the lower layers to harden, caused by the pressure above; this is known as *warehouse set*. Cement in this condition is in every way fit for service and can

be re-conditioned by letting each sack drop on a solid surface before using the cement contained.

Cement may be obtained in cloth or paper bags, in bulk, and in barrels. Cloth bags are the containers most generally used since manufacturers will accept the empty bags if returned in good condition. The consumer, however, must prepay the freight when returning the empty bags to the mill. The cloth bag will stand transportation, and its size and shape make it convenient to handle. If properly cared for, it may be used over and over again. The following is quoted from *Concrete-Cement Age*, May, 1913:

"Wooden barrels are out of the question because of increased cost of packing the cement into them; the large initial cost of the barrel itself (35 cents to 40 cents each); the inconvenience in handling a package weighing 400 lb., especially on large work; the increased cost represented by freight on weight of each barrel (approximately 25 lb.); and the increased cost of the cement to the dealer and the user because the barrel cannot be returned. It will be seen, therefore, that the use of the wooden barrel would entail a direct additional tax on the dealer or the user—all items considered—of approximately 40 cents per barrel.

"Paper bags are used to a limited extent. They cost the manufacturer 10 cents per barrel (4 bags to a barrel), and the additional burden to the dealer and the user is represented by this 10 cents per barrel, plus the loss resulting from breakage in transportation and handling, which is large (probably 5 cents per barrel), and very annoying. The shippers will not be responsible for damage to paper bags and, on the whole, paper bags which cannot be saved or returned impose an additional and wholly unnecessary burden of approximately 15 cents per barrel—all items considered—on the ultimate consumer.

"If a cloth cement sack can be used eight times—as it may be if properly cared for and returned promptly when empty—the loss or burden imposed upon the ultimate consumer from the use of cloth sacks is 5 cents per barrel only, which is only $\frac{1}{3}$ the burden imposed by the use of paper bags, and $\frac{1}{8}$ the burden imposed by the use of wooden barrels.

"Cloth sacks, cheaper than those now used for carrying cement, are out of the question, because they are too frail and weak to stand transportation and protect the heavy weight of cement."

Within the past year or two considerable cement has been shipped in bulk to cement-product factories and to construction jobs adjacent to railroad tracks. Much economy has resulted from the saving in labor, and from the elimination of package

losses and expense. There seems to be no difficulty in shipping bulk cement in tight box cars and this method will undoubtedly become more popular in the near future.

79. Aggregates.—It is important that great care be exercised in the selection of the aggregates for concrete—and this applies most particularly to the fine aggregate or sand. (See Appendix, Section B.)

The two most essential qualities to consider in sand are cleanness (that is, freedom from impurities) and the size of grain, or granulometric composition. The testing of the sand, by determining the tensile strength of mortar made from it, is the simplest way of proving its quality but this test should be supplemented by a sieve analysis, as explained in Art. 3 of Volume I.

The methods of tests for concrete materials are fully described in the Report of Committee on Specifications and Methods of Tests for Concrete Materials, National Association of Cement Users, presented at the Annual Convention at Pittsburgh, Pa., Dec. 10 to 14, 1912. This report which follows is very instructive and should be carefully studied.

The present report emphasizes the necessity for more rigid requirements for the acceptance of concrete materials and for routine tests of the materials and the concrete.

TESTS OF CONCRETE

Tests of the individual materials are not sufficient for determining the quality of the concrete made from them. A certain sand may give satisfactory results with one brand of cement and prove a failure with another. A sand of a certain granulometric composition may require an entirely different proportioning with one coarse aggregate than with another or may be unsuited for use with it because of the sizes of the grains.

The Committee recommends that for all important structures, such as dams, conduits, and buildings, specimens of concrete be made up in advance of the actual work from materials which it is proposed to use in the work. These specimens should be made under laboratory conditions so as to obtain uniformity in mixing and storage, and with the same consistency which is to be used in the proposed structure. The specimens should be stored in moist atmosphere and compressive tests made at the age of 28 days and also at other periods if practicable. The final selection of the materials and the proportions to be adopted should be governed by the results of these tests.

The tests now in progress in different laboratories under the auspices of the Committee are expected to furnish data from which definite recommendations may be made as to the size and shape of the specimen. The Joint Committee on Concrete and Reinforced Concrete recommend cylinders 8 in. diameter by 16 in. high. Where it is impracticable to adopt this size, a 6-in. cube or a cylinder 6 in. diameter and 6 in. high is tentatively suggested for use with aggregates under 1½ in. in size. It must be borne in mind that specimens of this shape may give from 10 to 20 per cent higher strength than the same materials made into oblong cylinders or prisms.

Layout of Concrete Tests.—In order to obtain well-defined information with reference to the effect of different shapes and sizes of specimens and different mixtures, the Committee has laid out in very full detail a number of series of tests. These specifications have been sent to various laboratories throughout the country and accepted by them as a part of their routine tests for the coming year. The results of these tests will be presented in the next report of this Committee.

TESTS OF CONCRETE MATERIALS

Tests of materials for use in mortar or concrete may be divided into two general classes:

(1) Tests for acceptance.

(2) Tests for quality.

These two divisions frequently merge into each other because in many cases the special characteristics of a sand may render it unfit for certain work, although it may satisfactorily pass a standard test for acceptance.

The most important questions to be asked in regard to a material are: (*a*) Can it be used? (*b*) In what proportions must it be mixed with the other materials to produce the required qualities in mortar or concrete? (*c*) Can another material of the same class be substituted with greater economy?

For small structures and, in fact, frequently for larger ones where the sources of supply are limited, the all important question is a combination of the first two, namely, can a certain material—and this applies most frequently to the fine aggregate or sand—be used, and in what proportions to give the desired quality of concrete or mortar? For structures where the volume of the concrete is large so that the economy of the proportioning is paramount, or for small structures subjected to special conditions—such as the effect of sea water—the cement and aggregates should be thoroughly studied also and special tests made to determine the most economical or the qualifications for the work proposed.

Taking the matter of the selection of sand alone, an illustration of the poor quality that is frequently used in practice is found in the tests in the laboratory of one of the members of the Committee. Out of 37 samples of sand from different parts of the country made into 1 : 3 mortar, 12 were equal to or greater in strength than similar mortar of standard Ottawa sand; 5 were between 90 and 100 per cent of standard sand; 6 were between 80 and 90 per cent; 4 were between 70 and 80 per cent; 6 were between 60 and 70 per cent; 1 between 50 and 60 per cent; 2 between 40 and 50 per cent; and 1 between 30 and 40 per cent. Such results as these indicate the absolute necessity for always testing sand that is used in concrete. For large and important structures, such as dams, tunnels, aqueducts, buildings, and other reinforced concrete work, not only preliminary tests should be made but frequent tests at stated intervals to avoid the danger of inequalities in the supply.

TESTS OF MATERIALS FOR ACCEPTANCE

The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association.

Fine Aggregate

Sand or other fine aggregate should always be tested. In ordinary cases where special characteristics are not required, the test for acceptance is the determination of the strength of mortar made up with the cement and fine aggregate to be used on the work in comparison with the strength of identical specimens made with standard sand. The results of this test also are an aid to the selection of the proportions which should be adopted.

Compression tests of mortar are the most reliable, but tensile tests may be employed when a compression machine is not available.

Selection of Sample.—Take an average sample aggregating 20 lb. Ship in strong box or bag or barrel, so that laboratory will receive material in its natural condition. Ship in a separate package from the cement.

Quartering.—Unless grains are of uniform size, separate from the large sample the quantity required for the test by quartering or other similar means.

Making Specimens.—Mold a large enough quantity so that there will be at least four specimens each to test at 72 hours, 7 days, and 28 days, in proportions 1 part cement to 3 parts sand by dry weight. Bank sand should not be dried, since drying may affect the quality,

but the percentage of moisture should be determined by test on another sample, and corrected for in making the weights. The consistency should be the same as that required for standard sand mortar. From 10 to 40 per cent more water may be required for bank sands or artificial aggregates than for standard Ottawa sand to produce the same consistency.

Similar specimens of the same cement mixed at the same time under the same conditions should be made with standard Ottawa sand in the same proportions.

All temperatures of mixing and storage should be maintained at 70° F. During the first 24 hours the specimens should be stored in moist air and the remainder of the period in water.

The 72-hour test is the most severe and sand failing to attain the requirement at this age frequently reaches it at 7 or 28 days and can be accepted. If, however, the 72-hour briquets break in the clips of the machine or if the specimens at this age show very low strength, say 25 per cent, or below, of the strength of standard sand mortar, the sand should be considered dangerous to use on any important work of construction.

Requirements for Strength.—The aim in concrete construction should be to obtain a sand which produces a 1:3 mortar equal in strength to a similar mortar of standard Ottawa sand. Sands producing mortars having a strength of less than 70 per cent of standard sand mortar should be rejected.

Between these two limits the variations in strength should be taken into account in fixing the proportions of the mortar or concrete.

Other Tests.—For work requiring special characteristics, the fine aggregate must be accepted not merely on passing the tests for strength, but other tests, such as the test for permeability, the effect of different brands of cement, the effect of frost action and the effect of fire. Characteristics of yield, density, chemical composition, granulometric composition, and amount of organic matter, are frequently required.

Coarse Aggregates

It is recommended that the quality of the coarse aggregate be determined from the preliminary tests of concrete made with the materials and in the proportions designed to be used in the work. Government tests made at the Structural Materials Laboratory in St. Louis under the direction of Mr. Richard L. Humphrey, clearly show that the hardness of the particles of coarse aggregate appreciably affect the strength of the aggregate, so that greater stresses may be used in concrete when the stone is hard than when it is soft. In certain cases the chemical composition of the coarse aggregate may affect the strength and durability of the concrete.

TESTS FOR QUALITY OF FINE AGGREGATE

In addition to the tests for acceptance which have been discussed in the preceding paragraphs, many aggregates require special tests Furthermore, various laboratories have adopted different classes of tests to use with aggregates which come to them for examination. This can be discussed best by a consideration of the practice in different laboratories.

During the past year this committee has been in communication with all of the laboratories of which they were able to learn, in an effort to gather as complete data as possible relative to the various tests for a sand now in use. Each laboratory was requested to furnish a complete description of their methods of testing sands—in most cases such descriptions were freely furnished. In all, 28 laboratories were communicated with and replies were received from 24; of these, 15 gave more or less complete descriptions of their tests.

The data in these replies were tabulated and listed and gave some very interesting information. It was found that some 26 different steps are taken in these laboratories for the determination of the suitability of a sand for use in concrete or mortar.

Listed and briefly described, these steps are as follows:

1. *Examination of Deposit* to determine its uniformity, stratification, extent, overburden, methods of stripping, handling, screening, washing, etc.

2. *Preparation of Sample* to eliminate stones, water, silt, and to insure its being representative of the bank or the quantity from which it was taken.

3. *Tensile Test of Mortar* to determine the strength of known mixtures of the sand and Portland cement at various ages and to compare this strength with that of standard Ottawa sand.

4. *Compressive Test of Mortar* to determine the strength of known mixtures of the sand and Portland cement at various ages and to ascertain what mixture is necessary to produce a required strength, or impermeability.

5. *Compressive Test of Concrete* to determine the strength of known mixtures of the sand and a coarse aggregate and Portland cement at various ages and to ascertain what mixture is necessary to produce a required strength or impermeability. In the case of bank gravel or crushed stone which is not to be screened and reproportioned this test is used instead of No. 4 above.

6. *Percentage of Moisture* is determined in order to allow for the water contained in the sand in proportioning mixtures, computing freight rates, etc.

7. *Percentage of Voids* is determined:

(a) By determining the specific gravity of the solid particles and

weighing a known volume of the sand and computing therefrom the percentage of voids.

(b) By filling the voids in a known volume of sand with a measured amount of a liquid.[1]

8. *Yield, or Volume of Mixtures* is determined by adding a known quantity of cement to a known quantity of the sand, wetting the mixture and noting the increase in the volume of the sand. This is repeated with various proportions of cement and sand to ascertain the amount of cement required to fill the voids in the sand.

9. *Density of Mortar and Concrete* is determined for various mixtures of cement and sand and of cement, sand, and stone to ascertain that mixture which gives a material of the greatest weight per unit of volume.

10. *Specific Gravity of Solid Particles* is determined by three different methods:

(a) By the specific gravity apparatus such as the Jackson flask.

(b) By pouring a known weight of sand into a known volume of water and noting increase of volume of liquid.

(c) For coarse aggregate by suspending pieces by a thread on chemical scales and noting weight in air and weight when hanging in a jar of water.

11. *Specific Gravity of Sand* including voids is determined by weighing a known volume of the sand exclusive of moisture and computing the specific gravity from the known weight of the same volume of water. Having determined the specific gravity of the solid particles (Test 10) and also that of the sand (including voids) the per cent of voids in the sand may be calculated.

12. *Weight per Cubic Foot* is determined by weighing a known volume of the sand. This test is very closely related to Test 11.

13. *Granulometric Analysis or Mechanical Analysis* is made by passing a known weight of the sand through a series of sieves of various sizes and noting the amount retained on each sieve and the amount passing the finest one. This test shows the distribution of the particles from fine to coarse.

14. *Uniformity Coefficient* is determined from a curve plotted from the granulometric analysis by dividing the diameter of the particles at the point where the curve crosses the 60 per cent ordinate by the diameter of the particles at the point where the curve crosses the 10 per cent ordinate.

15. *Wet Screening* of a sand on the 100- and 200-mesh sieves shows the amount of fine material which can be washed from the sand and which in the case of dry screening in Test 13 often adheres to the larger particles and so leads to uncertain results.

[1] This method as usually practiced is very inexact on account of the air entrained.

16. *Silt Suspended in Water* is determined by agitating a quantity of the sand with a large excess of water in a tall cylindrical glass vessel and noting the amount of silt which becomes suspended in the water and later settles down on the top of the sand.

17. *Silt Washed Out* is found by agitating a known quantity of the sand in water, allowing the coarse particles to settle and decanting or syphoning off the muddy water from the top. This operation is repeated until clear water is obtained. The amount of silt washed out is determined after evaporating the water.

18. *Loss on Ignition* is determined by heating to a red heat a weighed quantity of the dried sand and again weighing after cooling. This test is best performed only on the silt washed from a quantity of sand as the silt usually contains the injurious organic constituents of the sand.

19. *Organic Matter* is sometimes determined after the methods employed by the agricultural chemists.

20. *Chemical Analysis* is made to determine the character of the sand grains. Only a few of the most common rock constituents are usually determined quantitatively, for the purpose of judging of the strength and durability of concrete made from the sand. A high silica content is usually desired.

21. *Microscopical Examination* is made to ascertain the approximate size of the grains, their shape, character of surface and to detect the pressure of objectionable material or foreign matter such as mica or small roots or a coating on the grains.

22. *Absorption of Mortar and Concrete* made from the sand mixed in varying proportions with cement or cement and stone is determined to ascertain its suitability and the proper proportions for the production of a product having a minimum of absorption.

23. *Permeability of Mortar and Concrete* made from the sand is sometimes determined for various proportions of the constituents in order to determine mixtures for the production of an impermeable mortar or concrete.

24. *Effect of Different Cements* on some sands is quite variable and tests are sometimes made to determine with which of several available cements the sand will give best results.

25. *Freezing* during the setting time has a more injurious effect on mortar and concrete made with some sands than those made by others and tests are made to determine the extent to which the sand is affected by freezing.

26. *Fire* has a more disastrous effect on some sands when used in mortar or concrete than it has upon others and a heat test is made to show the ability of the sand to resist fire.

NUMBER OF LABORATORIES USING EACH TEST

Test	Number of laboratories
1. Examination of deposit	4
2. Preparation of sample	8
3. Tensile test of mortar	15
4. Compressive test of mortar	7
5. Compressive test of concrete	5
6. Per cent of moisture	7
7. Per cent of voids	15
8. Yield or volume of mixture	3
9. Density of mortar and concrete	2
10. Specific gravity of solid particles	4
11. Specific gravity of sand	5
12. Weight per cubic foot	2
13. Granulometric analysis	16
14. Uniformity coefficient	4
15. Wet screening	1
16. Silt suspended in water	4
17. Silt washed out	8
18. Loss on ignition	5
19. Organic matter by agricultural methods	1
20. Chemical analysis	5
21. Microscopical examination	5
22. Absorption	1
23. Permeability	3
24. Effect of different cements	2
25. Freezing	1
26. Fire	1

No doubt other tests of fine aggregate are in use which have not come to the notice of the Committee, but the very large range and lack of uniformity is illustrated by the outline given. No two of the laboratories used the same tests, and scarcely two of them carry out their tests in exactly the same manner, so that at present it is difficult or impossible to accurately compare the results obtained in one laboratory with those obtained in another.

Examination of the list shows that the popular tests for sand at the present time are granulometric analysis, tensile tests of mortar, and percentage of voids. Of these only one, the tensile test of mortar, belongs properly in the classification of tests for acceptance as discussed in preceding paragraphs of the report.

The problem before the engineer who is interested in proper methods of testing materials is to select and standardize such tests as will give the necessary data which will determine whether or not the sand is suitable for a particular use and, if suitable, in what proportions it should be used to produce desired qualities in the resulting mortar or concrete.

Impurities in sand and stone may usually be removed by washing. The washing for small jobs may be done in almost any type of batch concrete mixer by simply removing the mixer and letting the hose play inside without stopping. When the mixer overflows, the water will carry off the foreign matter. For larger jobs, however, the most satisfactory plan is to wash the materials down a trough over screens, or against and through screens inclined in the opposite direction from the trough. Screens with round-punched holes are better for this purpose than wire mesh.

80. Metal Reinforcement.—The requirements and tests for steel are fully explained in Chapter II of Volume I and in Sections B and H of the Appendix. Types of reinforcement have been described in Chapter V.

CHAPTER XIX

FORMS

The design, construction, and erection of forms should be given careful attention since they are matters of the greatest importance as regards the cost and satisfactory execution of reinforced-concrete work. In fact, the cost of forms frequently determines the amount of profit or loss in the building of reinforced-concrete structures.

81. General Requirements.—In general, form material and workmanship should be such that the finished concrete will be true to line and surface, with no irregularities, and great care should be taken to obtain tight joints in order to prevent leakage. Leakage produces either sand or stone pockets on the surface of the finished concrete and gives the structure a bad appearance.

Economy in form work depends to a considerable extent on many of the small details in design and construction. These details should be such that the forms may be easily removed and so that they may be used over and over again, as there is seldom economy in making rough forms which must be ripped to pieces when taken down. Square corners in forms should be avoided, and this may be accomplished, in the case of re-entrant angles, by beveling the sheathing lumber; in other cases, by inserting triangular pieces, usually called V-strips or bevel strips. This practice improves the appearance of structural members and makes form removal easy. Where slab panels meet girder forms, a good plan is to place triangular strips across the ends of the slab boards in order to prevent the end grain of the wood showing on the finished concrete.

Forms should be durable and rigid, and with sufficient strength to carry the loads that are likely to come upon them. Horizontal members should be capable of supporting the weight of the concrete and the construction load. Vertical members should be proportioned for the pressure of wet concrete.

Simplicity in form work should be considered in the design of the structural members. Where possible, uniform story heights

should be selected in order to prevent continual remaking of column forms, and frequent changes in column sizes should be avoided (at least in the case of light-floor construction), not only on account of the columns themselves, but on account of the beams and girders which frame into them. Also where it is feasible, the sizes of beams should be arranged so that the contractor may use his forms many times, thus greatly reducing the expense for lumber.

A slight excess of concrete may sometimes save three or four times the cost of same in carpenter work and lumber. Frequently beams may be designed of such widths as to use dimension widths of lumber without splitting.

82. Lumber for Forms.—The character of the work and the local market will generally govern the selection of the lumber to use for forms. The hardwoods, of course, are too hard to work and are too expensive. Of the softer woods which are cheap enough and of a satisfactory character for use in form work, spruce, fir, Norway pine, and southern pine are usually the most available. White pine is the best material to use where exceptionally smooth surfaces are required, but it is usually too expensive except for forming moldings or other ornamental designs.

Spruce, in sections where it is readily obtained, is perhaps the best material for joists, studs, and posts. A good quality of North Carolina pine, however, is preferable to spruce for panels because it is less likely to warp and has a harder surface.

Partly dry lumber is better for form work than either very green or kiln-dried lumber. Kiln-dried lumber swells and crushes or bulges the joints when wet concrete touches it. Very green lumber, on the other hand, does not swell enough to close the joints, and shrinks if not kept wet.

Undressed lumber may be used for the backs of walls, for work below ground, and wherever a smooth and true surface is unimportant. Lumber dressed on one side and both edges, however, is generally preferred by contractors even for rough work, because of the smaller cost of removing and cleaning. For floor and wall forms the lumber should always be dressed in this manner. It is still better to use lumber dressed on all four sides because of the convenience in handling, working up, and placing, which more than offsets the extra first cost.

The joints in form work deserve particular attention. Square or butt joints are generally employed in forms for columns, beams,

and girders, and, to make the joints tight, the lumber must be dressed true to edge. In floor and wall forms tight joints may be obtained by using either tongue-and-grooved stock or stock cut with one edge beveled and the other square. Beveled-edge stuff is cheaper than tongue-and-grooved, but does not give as satisfactory results when employed in forms which are to be used over and over again. Under such conditions the tongue-and-grooved stock holds its place better and gives smoother surfaces. The idea of beveling is to prevent buckling since the edges crush as the boards swell.

The thickness of lumber to use in form work depends on the number of times the forms are to be used and, in the case of slab and wall forms, on whether the boards are to be built into panels or nailed to their supports at each resetting. Generally speaking, 1-in. stock ($\frac{7}{8}$-in. dressed) is employed for floor and wall sheathing and for the sides of beams and girders; 1 to $1\frac{1}{2}$-in. stock ($1\frac{3}{8}$-in. dressed) for columns; and 2-in. stock ($1\frac{7}{8}$ or $1\frac{3}{4}$-in. after dressing) for the bottoms of beams and girders. If floor and wall forms are to be used many times (say, more than six times), especially if the lumber is very soft or is not built into panels, 1-in. stock is too thin and $1\frac{1}{4}$ or $1\frac{1}{2}$-in. should be used instead. Also, if the beams and girders are narrow, $1\frac{1}{2}$-in. stuff usually gives satisfactory results.

The heavier sheathing has a greater salvage value on the completion of a piece of work, but, if the forms are not to be used more than five or six times, this is offset by the fact that the lighter stock is easier to patch, not so expensive to handle, and is cheaper in first cost. Although the thicker material permits a greater spacing of the studs or joists, the saving here is not great since the quantity of lumber in these members is apt to be governed by the strength required to resist either the construction load or the pressure of wet concrete.

It is not always economical to use old lumber. The cost of taking apart small sections of forms may be greater than the actual cost of new lumber, and the expense of building is usually greater and less satisfactory. The cost of making over old forms under ordinary conditions has been found by observation to be about 90 per cent. greater than making up the same forms from new lumber.

83. Removal of Forms.—The length of time that forms should remain in place depends upon three factors: (1) the weather;

(2) the character of the members; and (3) the relation of the dead to the live load.

As is well known, the weather greatly affects the setting and hardening of concrete. The warmer the weather, the more rapid the hardening. Cold weather, on the other hand, retards hardening and thus prevents early removal of forms. If concrete freezes, it will not begin to harden until it has thawed, and then it will acquire strength very slowly.

The forms for members subjected to bending should be left in place longer than forms for columns or walls. Vertical members will bear their own weight when quite green, while horizontal members, such as floors, must harden until they can carry the construction load.

The greater the proportion of dead to live load, the longer the forms should remain. For example, the forms for an overhanging cornice should remain in place longer than the forms for almost any other member, since the dead weight is a very large percentage of the total weight which it is designed to carry. By a similar reasoning, long span beams or slabs should be supported a longer time than short ones; roofs longer than floors; columns in the upper stories longer than columns in the lower stories; and columns in general longer than footings.

Many large contractors mold a cube of concrete for each day's work and allow it to set out-of-doors under the same conditions as the construction work. When it is desired to remove the forms, these blocks are crushed in a testing machine in order to determine whether the concrete is strong enough to carry the construction load. Good judgment should be exercised even under such conditions since the plan does not provide for the possibility of an occasional poor batch of concrete.

Many builders have definite rules for the removal of forms under ordinary conditions. Such rules, of course, can only be roughly approximate, and knowledge and judgment are necessary in using them. As a guide to practice, the following rules are suggested in "Concrete Costs" by Taylor and Thompson:

Walls in Mass Work.—One to three days, or until the concrete will bear pressure of the thumb without indentation.

Thin Walls.—In summer, two days; in cold weather, five days.

Columns.—In summer, two days; in cold weather, four days, provided the girders are shored to prevent an appreciable weight reaching the columns.

Slabs up to 7-ft. Spans.—In summer, six days; in cold weather, two weeks.

Beams and Girder Sides.—In summer, six days; in cold weather, two weeks.

Beam and Girder Bottoms and Long Span-slabs.—In summer, 10 days or two weeks; in cold weather, three weeks to one month. Time to vary with the conditions.

Forms should be so designed that the column forms may be removed without in any way disturbing the supports of the beams and girders. A defect may then be detected and remedied before any load is brought to bear upon the column. Under ordinary conditions, column forms are taken down in two or three days after pouring, and therefore are generally ready for use on the floor above, even if the rate of construction is as rapid as a story in six or seven days. As soon as the concrete of the slabs is hard enough to allow the removal of the slab forms, the beam and girder sides are also taken down, leaving the beam and girder bottoms with their props in place for some time longer. Walls are generally built independently of the rest of the building and a wall form may be removed whenever the concrete in the wall is hard enough to bear its own weight.

84. Number of Sets of Forms.—The number of sets of forms required in the construction of a building depends upon the speed of construction desired, the shape of the building, and the weather. Under good conditions, 1½ sets of forms will allow the work to progress at the rate of a story in a week or 10 days. Some contractors, however, use only one set of forms in a building no matter how high, and provide additional lumber for beam and girder bottoms and supports which must be left in place.

In a building of small floor area, two sets of forms may be needed to permit the work to progress rapidly enough. On the other hand, forms for only about three-fourths of one floor may be needed if the floor area is exceedingly large. In the case of low basements or heavy columns, the basement forms cannot always be remade economically for use in the upper stories and an extra set of forms will be required under these conditions.

85. Examples of Form Design.—To get a general idea of form construction, Figs. 339 to 342 inclusive should be carefully studied.

Column Forms.—Figs. 328 and 329 show two typical forms for rectangular columns. In Fig. 328 the two sides of the column are

held together, as shown, by bolts, and the two opposite sides by hard-wood wedges between the bolt and the form. In Fig. 329 wedging strips are nailed to the longer cleats. Wood wedges serve as clamps for two sides of the column and iron clamps hold the other two sides. The column form represented in Fig. 328 is also shown in Figs. 330 and 379; the column form of Fig. 329 is the form used in the Kraus Building, Memphis, Tenn., Fig. 331. Iron clamps were employed without wedging strips in the

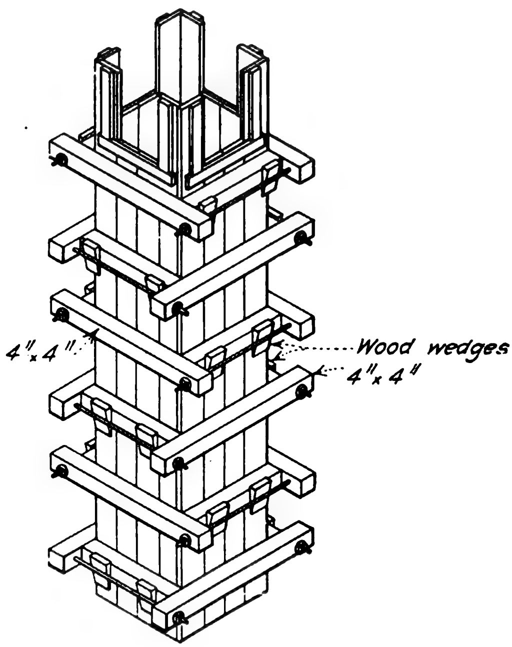


FIG. 328.

Hollender Company's Warehouse, Chicago, Ill., Fig. 332. Notice, however, that the vertical sheathing is of necessity connected by thin cleats or battens into panels to make it possible to hold it in place by the form of clamp used. This panel construction makes a unit which is more convenient to handle, but less convenient to adapt to changes in size of column. The salvage value of the lumber is also reduced by the nailing.

The corners of square or rectangular columns should always be chamfered to prevent the danger of breaking off these corners when removing the forms, and also for the sake of appearance.

The bevel strips should be nailed on to two opposite sides of the column forms when they are being made up.

The sides of column forms are usually made up of narrow strips of sheathing in order to facilitate the reduction in size of the columns from floor to floor. This is not true of exterior or wall columns since they are usually of the same width from basement to roof, and change but little in thickness.

Column forms should always be constructed with an opening at the bottom in order that the reinforcement may be adjusted and sawdust and other material cleaned out. The piece of board or

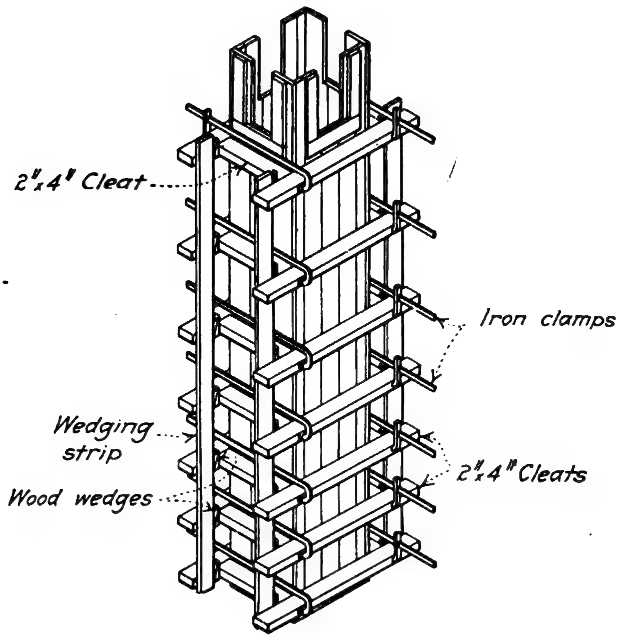


Fig. 329.

small panel cut off should be placed against the column form so that it will be all ready to put back in place before the concrete is poured.

A typical wall column detail is shown in Fig. 333 and is another type of column form design.

A design for an octagonal column form is shown in Fig. 334. There are many methods of making forms for this type of column, but the one shown is used to a considerable extent. Fig. 335

shows a method of constructing an octagonal form by means of chains and wedges.

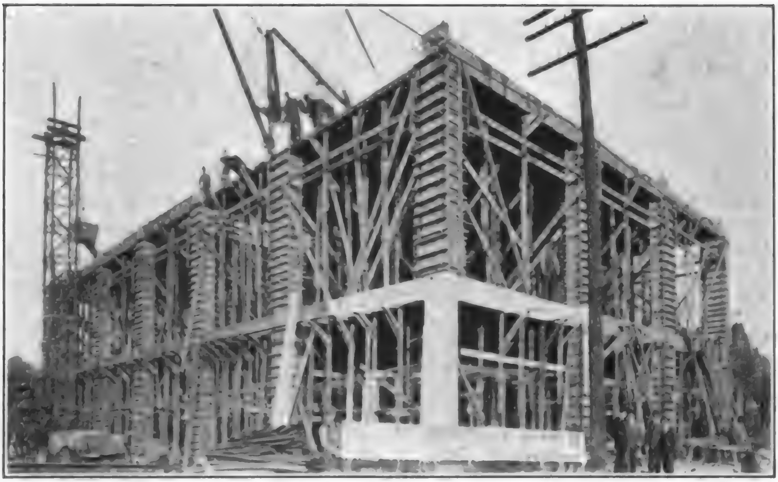
Beam and Girder Forms.—Fig. 336 is an isometric view of a

Courtesy of Mr. H. F. Porter, Mfg. Editor of "Factory."

FIG. 330.—Typical form work.

typical floor. Notice the supporting posts, also the *joist bearer* for supporting the joists under the slab form.

Figs. 337 and 338 show two methods of clamping beam forms together, but sometimes nailing is resorted to as shown in Figs.



Courtesy of Ferro-concrete Construction Co.

FIG. 331.—Kraus building, Memphis, Tenn.

Courtesy of Universal Portland Cement Co.

FIG. 332.—Hollender Company's warehouse, Chicago, Ill.

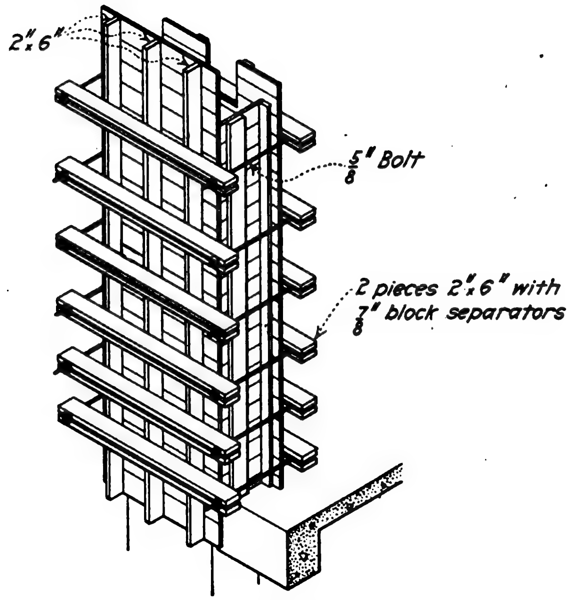


Fig. 333.

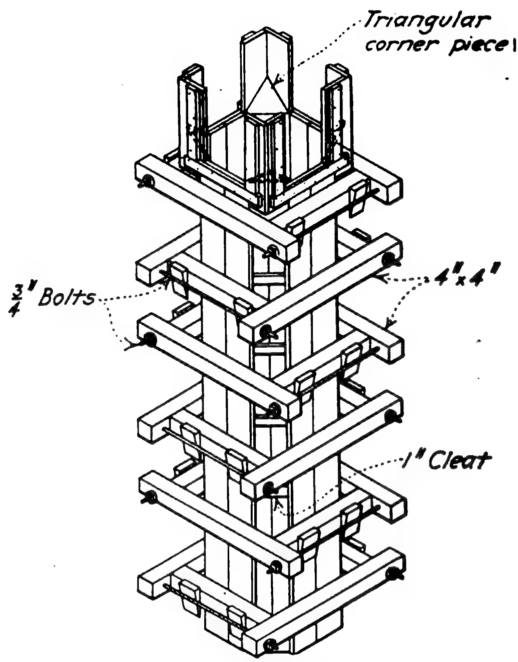


Fig. 334.

Courtesy of Universal Portland Cement Co.

Fig. 335.—Octagonal column form, Rogers & Hall building, Chicago, Ill.

339, 340, 341, and 342. The use of separate blocks instead of a joist bearer should also be noted.

Typical girder form construction is shown in Fig. 343. When the beams intersecting the girder are about the same depth as the

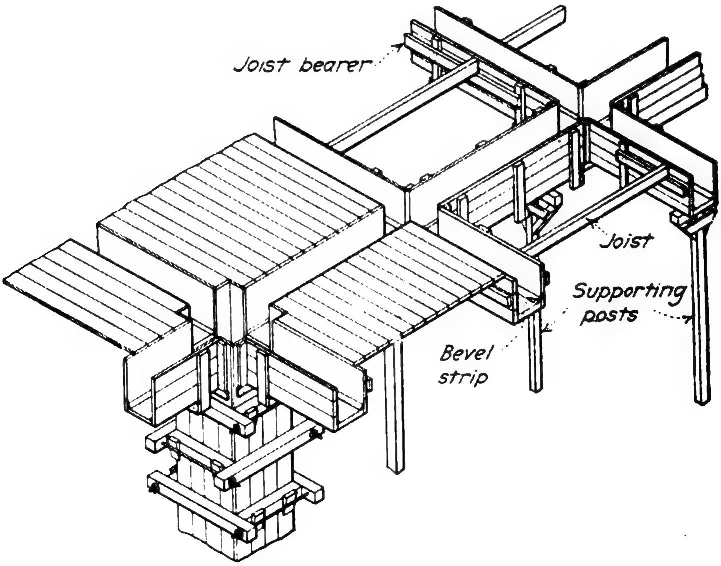


Fig. 336.

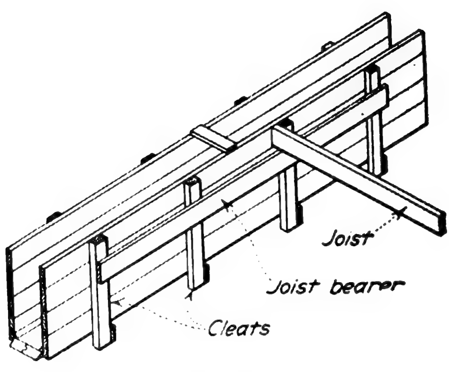


Fig. 337.

girder, the beam pockets should either be cut after the forms are in place or the sides should be made in sections between beams. The first method seems preferable.

A common method of bracing the outer side of a wall girder is shown in Fig. 344. Another method of bracing by means of a cantilever brace is shown in Fig. 330. Note the character of the lintel post which is shown again in Fig. 345. A method of haunch design is shown in Fig. 346.

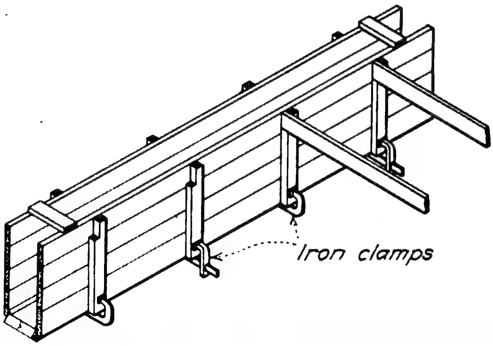
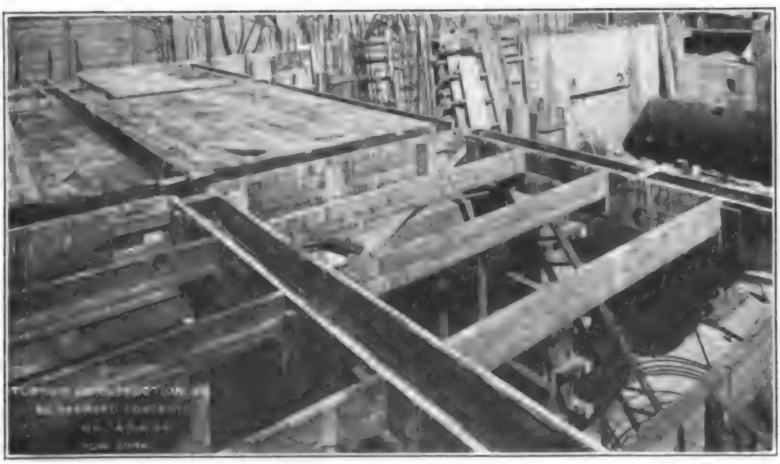


FIG. 338.



Courtesy of Turner Construction Co.

FIG. 339.—G. B. Seeley Son's factory, 15th Street, New York City. Note floor and girder forms and 2-in. by 6-in. joists.

Slab Forms.—Two types of slab forms need to be considered for ordinary construction where beams and girders are employed: (1) sheathing made up into panels with thin cleats (the only type of form construction so far treated), and (2) slab forms made up

Courtesy of Universal Portland Cement Co.

FIG. 340.—Successful Farming Publishing Co.'s building, Des Moines, Iowa.

Courtesy of Universal Portland Cement Co.

FIG. 341.—Successful Farming Publishing Co.'s building, Des Moines, Iowa.

into box shape comprising the joists and the sides of the beams and girders.

Courtesy of Universal Portland Cement Co.

FIG. 342.—Successful Farming Publishing Co.'s building, Des Moines, Iowa.

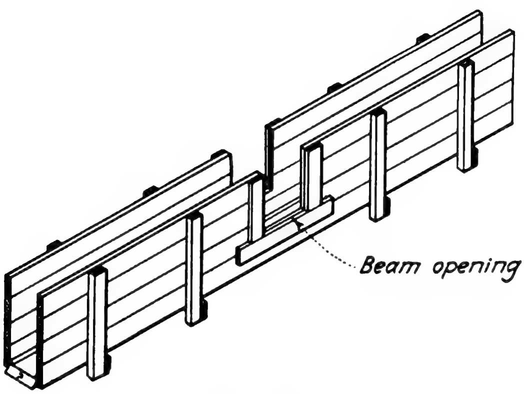


FIG. 343.

In general, slab forms are usually made in two, three, or four sections depending upon the dimensions and shape of the panel.

One-section forms bind at the sides and ends after having been wet by the concrete and, unless the beam and girder sides are flared and a type of form used such as is shown in Fig. 45, it is a difficult matter to pry the forms loose after the concrete has

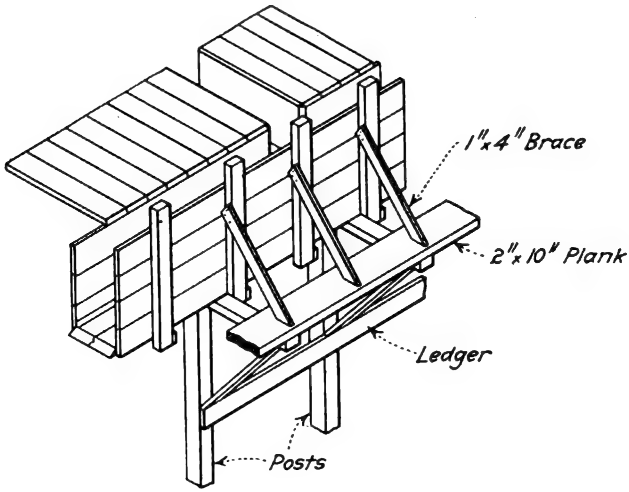


FIG. 344.

hardened. Four sections, with the joints at right angles, is the best method for easy removal.

In the usual type of construction, designated above as type 1, the beam and girder forms are put together on horses on the floor

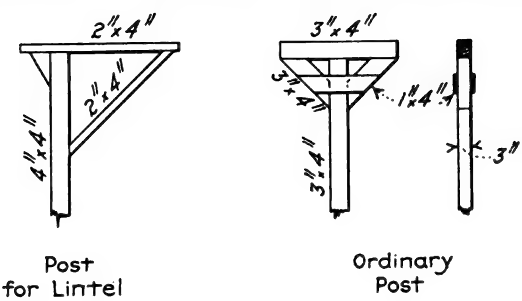


FIG. 345.

below and are then raised to place as one member. The joists are then put in position and the slab panels are placed on top of them without nailing. Where necessary, the slab panels are cut out at the corners for the columns. When the forms are to be

used but once, the slab sheathing is not made up in advance into panels, but is lightly nailed to the joists after they are in place.

Fig. 347 shows one method of making up slab forms into box shapes. In this design the entire sides of the beam and all but

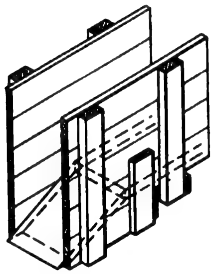


FIG. 346.

the bottom board of the girder sides are included in the slab forms. Metal strips are used to cover the joints between the sections. The size of a section is governed to some extent by the

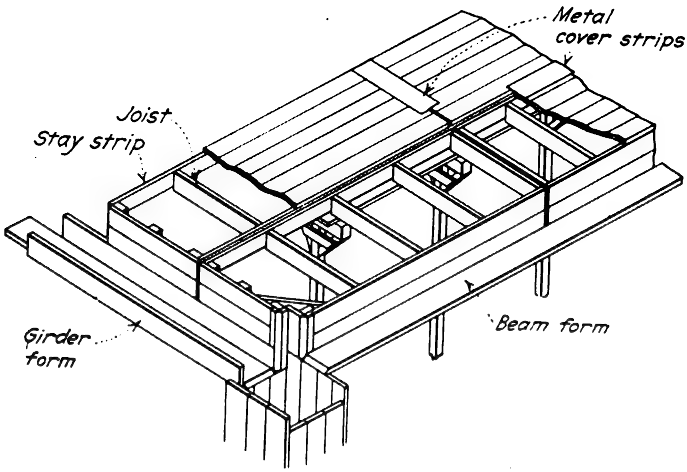


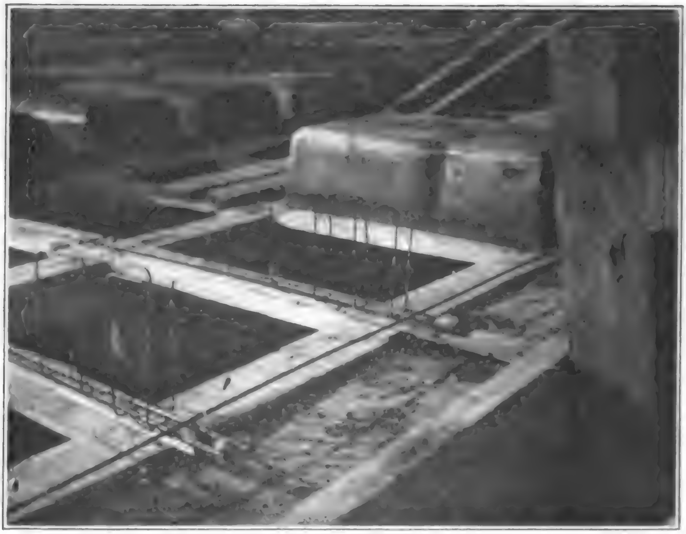
FIG. 347.

weight, since a form unit should be such that four men can readily handle it.

The form construction used at The University of Wisconsin (Art. 17) is shown in Fig. 348. Planks extend in two directions

supporting the cells. The steel and continuous stirrups are set in place for demonstration only.

A collapsible core box is used by at least one prominent builder (Fig. 349). A plank resting on cleats on the sides of the cores



Courtesy of Mr. Arthur Peabody, Supervising Architect, The University of Wisconsin.

FIG. 348.—Box forms used at The University of Wisconsin.

forms the bottom of the beam mold. The main girders are molded in similar spaces between the ends of the cores in one panel and those in the next panel. The molds are made in two equal parts with a hinged joint through the longitudinal center line of

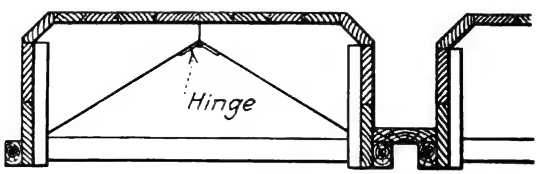


FIG. 349.

the upper surface and are held open by transverse struts between the lower edges. When the forms are to be removed, the transverse struts are knocked out and the boxes are collapsed. The use of these molds presupposes a standardized layout.

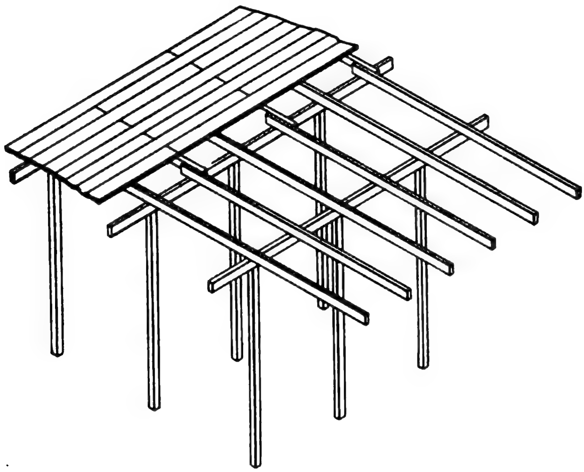


Fig. 350.

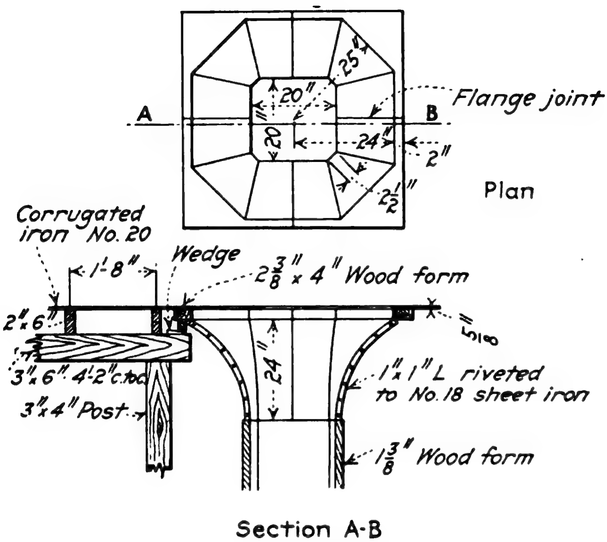


Fig. 351.

Forms for flat-slab floors are much simpler than for the ordinary beam and girder construction. Although enlargement of the column heads in such construction considerably increases the cost of the column forms, this is generally more than offset by the saving in the forms for the slab. A typical design of forms for flat-slab floors is shown in Fig. 350. The posts support stringers in one direction, upon which are joists to take the panel sections. Corrugated iron has been used to a considerable extent for sheathing where the ceiling surface does not need to be absolutely smooth (Figs. 112, 159, and 373). Plain sheet steel is not suitable for this purpose since it tends to warp and dent in

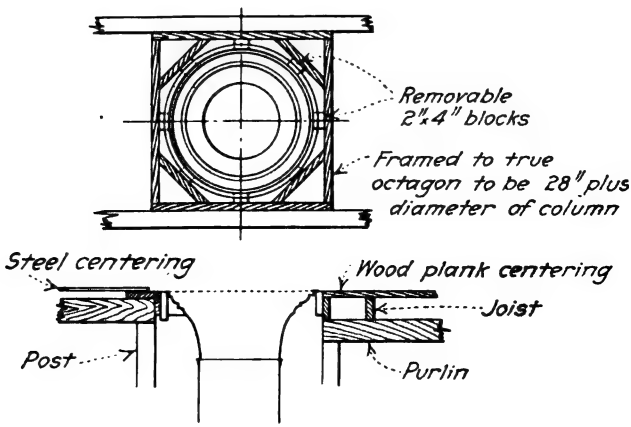


Fig. 352.

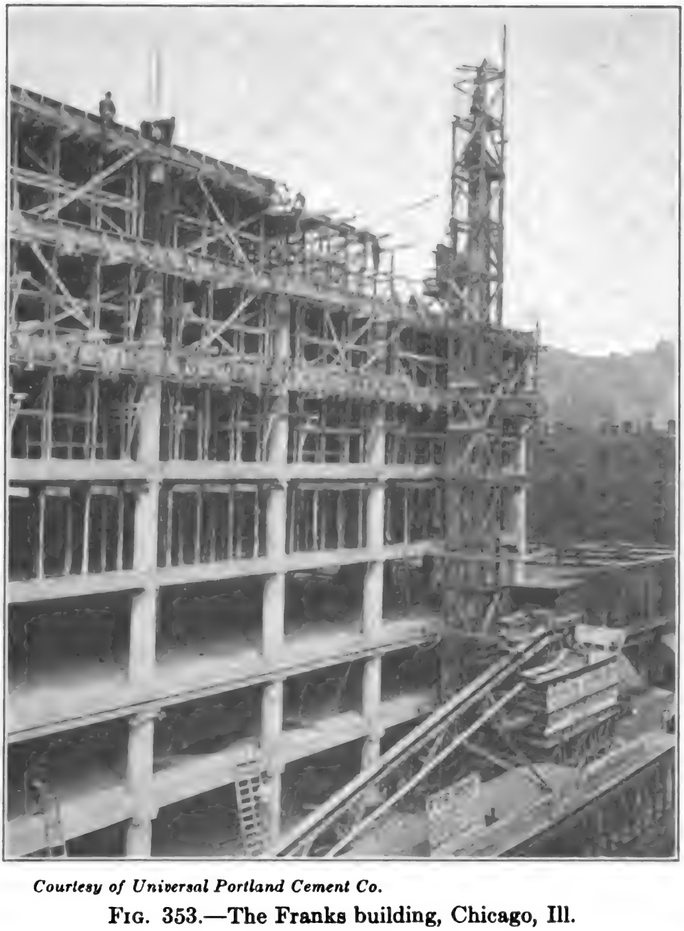
handling. Corrugated metal holds its shape better and can be straightened after each removal by running it through rolls.

There are many other types of floor forms but those described above illustrate the main features in floor-form design.

Column Heads.—The problem of the column head in flat-slab construction has been met practically from the first by the use of metal in some form. The metal mold used by the Aberthaw Construction Co. in the Massachusetts Cotton Mills at Lowell, Mass., is shown in Fig. 351. Note that the head is very simple, a square where it leaves the column, with flaring beveled corners, which finally form an octagon at the ceiling level. A similar column head is shown in Fig. 228.

Fig. 352 shows a type of column-head form used in constructing the Larkin warehouse at Chicago, Ill. The columns them-

selves were molded by steel forms and these forms were placed after the erection of the floor falsework. The column-cap section was suspended from the floor forms and the remaining column sections bolted to it. A suspended column-cap section is shown in Fig. 353. Fig. 354 shows such a form in place.



Courtesy of Universal Portland Cement Co.

FIG. 353.—The Franks building, Chicago, Ill.

Wall Forms.—Fig. 355 shows a common type of form construction for curtain walls, and Fig. 356 shows a similar construction for curtain walls below windows. Curtain walls under construction may be seen in Fig. 357. Fig. 358 illustrates the form construction for a heavy wall where the ties consist of wire twisted

to hold the sides of the forms in place. A simple form of construction for a low foundation wall, such as is employed for residences, is shown in Fig. 359. Fig. 360 illustrates a common method of building a wall of considerable height by means of movable forms. As soon as one section of a wall is completed,

Courtesy of Universal Portland Cement Co.

FIG. 354.—Flaring column heads. Hollender Company's warehouse, Chicago, Ill.

the bolts were loosened, and the slotted clamp allows the form to be moved upward.

Cornice Forms.—A typical form construction for a concrete cornice is shown in Fig. 361. The construction view in Fig. 362 should also be noted.

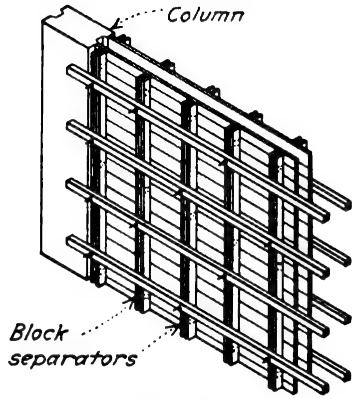


FIG. 355.

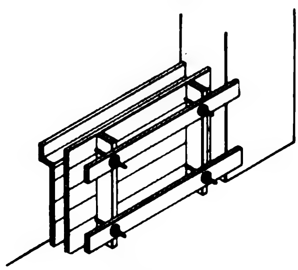
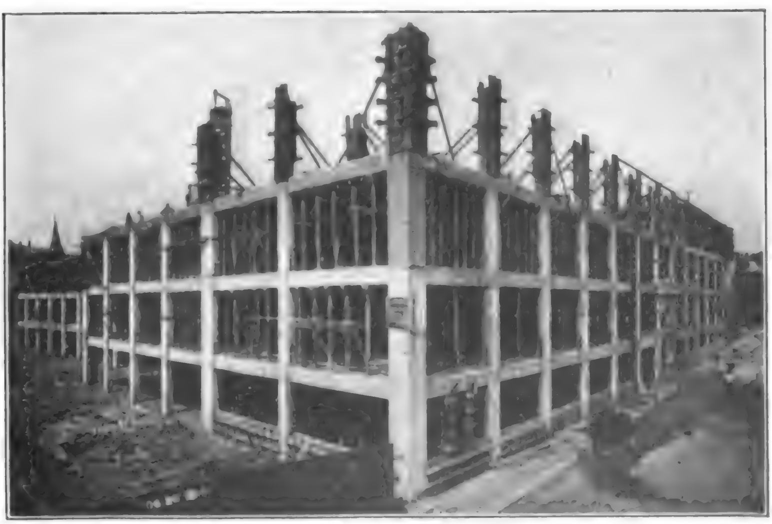


FIG. 356.



Courtesy of Densmore & LeClear.

FIG. 357.—Construction view. Shoe factory for Albert M. Lyon, Roxbury, Mass.

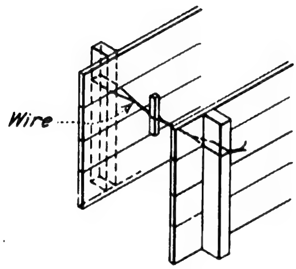


Fig. 358.

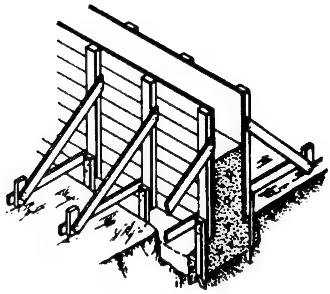


Fig. 359.

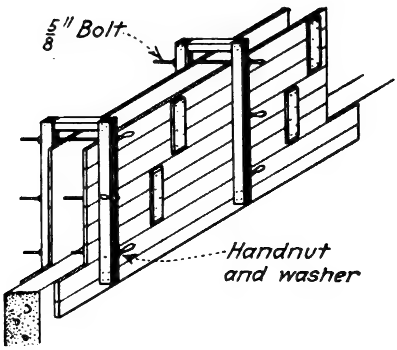


Fig. 360.

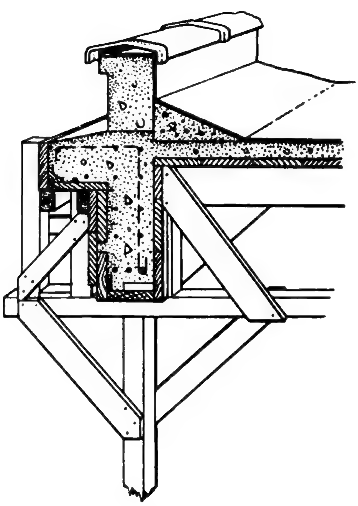


Fig. 361.

82. Design of Forms.—Although form design in most cases has been left entirely to the discretion of the superintendent or foreman on the job, it has been found by a number of engineering contractors that it pays on large or important work to have the forms designed in the drafting room, provided such designing work is done under the direction of a man of practical ability. This plan is not only more economical, but also eliminates the possibility of a superintendent making an error in judgment

Courtesy of Turner Construction Co.

Fig. 362.—Sugar and coffee warehouse, Brooklyn, N. Y., Arbuckle Brothers, 300 lb. floor load. Construction view showing the form work for heavy cornice.

(which may be serious) in a place where computation of strength is the only means of determining the proper spacing of supports. If form drawings are made, the lumber may then be ordered direct from the mill to exact widths and even-foot lengths, and, in cases where forms are of fairly uniform sizes, it is possible and sometimes of great advantage to order the lumber to the exact lengths and widths required.

The thickness of sheathing to use in different parts of the work, and many other such details of form design, depend almost entirely upon judgment and experience; but, on the other hand, the size and spacing of supports for such sheathing in slab, column, and wall forms and the spacing of posts for beams and girders are capable of being determined by scientific calculation and such practice results in the use of a minimum quantity of lumber consistent with the deformation allowed. In sheathing, joists, studs, and column clamps, the stress due to bending should not be greater than the allowable with a maximum deflection of $\frac{1}{8}$ in. In no case, however, should the spacing of the supports be greater than a safe span for the sheathing. Deflection should always be considered in order to give sufficient stiffness to prevent partial rupture of the concrete and to prevent sagging beams.

In calculating floor forms, we must add to the weight of the concrete itself a live load which may be assumed as liable to come upon the concrete while it is setting. This live load is generally taken at 75 lb. per square foot except in cases where the floor is made a storage for cement or sand.

The pressure of concrete against forms depends principally upon the rate of setting, the rate of filling, and the temperature. The following table is from tests made by Major Francis R. Skunk, U. S. A.:

PRESSURE OF CONCRETE ON FORMS
(Pounds per Square Foot)

Rate of filling vertical feet per hour	Temperature				
	80°	70°	60°	50°	40°
2	530	560	600	680	790
3	690	720	810	920	1080
4	820	870	980	1130	1340
5	930	990	1120	1310	1570
6	1020	1090	1250	1480	1780
7	1090	1170	1350	1620	1970
8	1130	1240	1440	1740	....

Sheathing and supports for the same may be figured for bending by the formula $M=\frac{wl^2}{10}$ unless it is quite clear that simply-sup-

ported action exists. Mr. Sanford E. Thompson recommends that the deflection of forms be computed by the formula $D = \frac{3}{384}\frac{wl^3}{EI}$. This formula is the ordinary one for calculating deflection except that the coefficient is taken as an approximate mean between $\frac{1}{384}$ for a beam with fixed ends and $\frac{5}{384}$ for a beam with ends simply supported. For lumber commonly used in form construction E may be assumed at 1,300,000.

The following is taken from the *Engineering Record,* issue of September 7, 1912 and shows the method of systematizing form work practiced by the Aberthaw Construction Company of Boston:

"In the construction of the forms for building the 10-story warehouse of the Larkin Company, Buffalo, N. Y., the Aberthaw Construction Company of Boston employed methods which are worth notice. It was found that it paid to use considerable refinement in the drafting of forms in the offices and to systematize the receiving of lumber and disposing of forms from the saw mills at the job. The form lumber was detailed at the Boston office just as steel would have been, the policy of this company being to pay thinking men to think and to pay carpenters to saw boards. In order to understand that which follows it should be noted that the Larkin building is 580 ft. long by 109 ft. wide, the floor space being over 14 acres, bounded by Van Rensselaer, Carrol, Exchange and Hydraulic Streets.

"At the home offices of the Aberthaw Construction Company a location plan of all beams was drawn, in which similar beams were given identical numbers, irrespective of location in the building itself. From this location plan, form drawings in detail were made, a detail drawing being required for each different sized form, both for sides and bottoms of beams. All different varieties of column forms were also detailed.

"In the system of lettering used on the Larkin job the Aberthaw Company suffixed letters to the beam numbers to show the exact location of each form in the building. As stated above, the building proper is bounded by four streets, and any form of a certain beam facing one of these streets had in its symbol the cpapital letter which stood for the name of the street. For instance, a form for beam No. 22 might be marked BSV22, meaning beam side, Van Rensselaer being the name of the street, and 22 the number of the beam itself. The bottom form for this same beam would be designated BBV22.

"In regard to the column forms, an additional number had to be

added to the symbol, as there is a variation in the height of floors. The figure 1 prefixed to the ordinary column symbol indicated that the particular form was for the first story. In a like manner the figure 2 showed that the column form was for the second story. To illustrate, a column form with symbols 2CSV25 meant second-story form column side Van Rensselaer street, while 25 was the number of the column proper as designated by the original location plan.

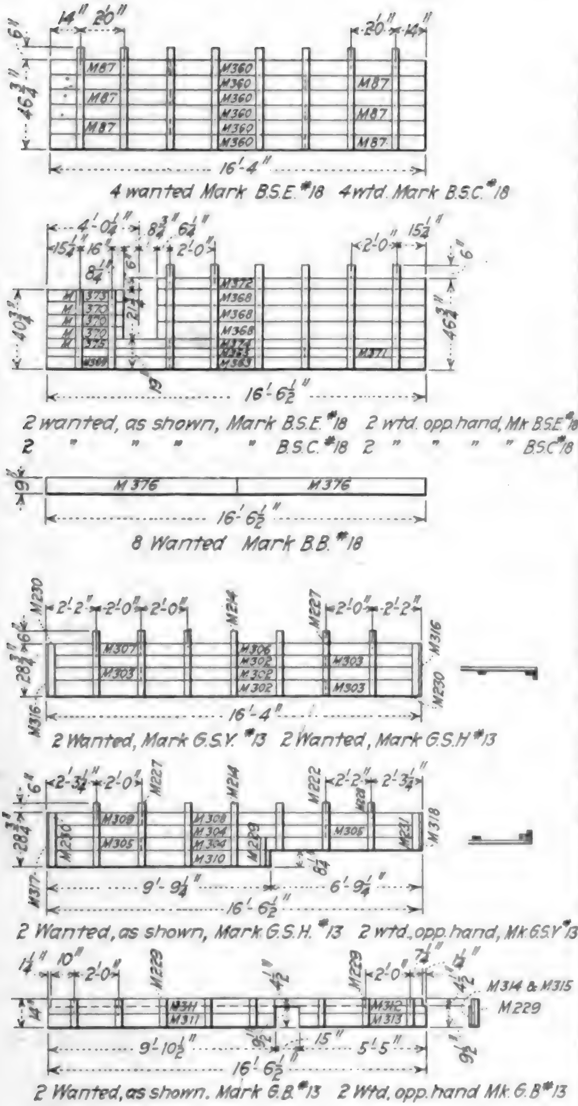
"As the height of stories varied, form details were designed complete for two stories, and to each form detail was given a symbol to be sten ciled on this form when made up, together with the number or quantity required in the first two stories of the job. On this same form detail the width of the form was divided into the exact number of boards which were to make up this total width of the dimensions written on the drawing. Therefore, having given the exact length of form and the width of the different boards going to make up this form and the total number of forms required for the job, it is readily seen how the sum total of all boards was easily obtained.

"Aside from these form details it was necessary to draw plans showing assembling of forms, the chief object of this being to show spacing of posts and jacks, amount of bracing and location. From this assembly plan, together with the layout plan of the building, the exact number of pieces and size of lumber constituting the centering could be scheduled. On this completed schedule bids were obtained from various lumber dealers before ordering.

"Inasmuch as the total number of board feet of lumber was nearly 1,000,000, it was necessary to adopt some systematic plan of piling and laying out the lumber. The chief attention was given to the dressed stock going to make up the forms proper since this stock was taken to and from the mill to the benches. It was necessary so to pile it that when a laborer wished a certain number of boards of a certain size he could find them quickly. A plan was therefore drawn showing the lumber layout and the relative location of different piles of lumber according to widths and lengths. Signs were placed in front of each individual pile stating the dimensions.

"On this job the Aberthaw Company found it was convenient and economical to build two mills since 90 per cent of the dressed stock had to be cut to exact length. Both mills had practically identical equipment except for layout. The most used piece of machinery in both mills was the cutoff saw, on either side of which was a table where the stock to be cut was placed. In addition to the cutoff saw each mill was equipped with a rip saw, boring machine, planer, and emery wheel.

"A clear understanding of the methods employed to make up a complete form from the blueprint, as received from the Aberthaw Company's Boston office, may be obtained by following in detail a



LUMBER SCHEDULE

1ST & 2ND STORIES

Wall Beam #18

No. Pcs.	Size	Length	Mark
48	$1\frac{1}{4}" \times 7\frac{3}{4}"$	$11'-2"$	M360
48	$1\frac{1}{4}" \times 7\frac{3}{4}"$	$5'-2"$	M87
24	$1\frac{1}{4}" \times 7\frac{3}{4}"$	$11'-9\frac{1}{2}"$	M368
8	$1\frac{1}{4}" \times 7\frac{3}{4}"$	$11'-3\frac{1}{4}"$	M363
8	$1\frac{1}{4}" \times 7\frac{3}{4}"$	$5'-3\frac{1}{4}"$	M369
24	$1\frac{1}{4}" \times 5\frac{3}{4}"$	$3'-3\frac{1}{2}"$	M370
8	$1\frac{1}{4}" \times 5\frac{3}{4}"$	$11'-3\frac{1}{4}"$	M365
8	$1\frac{1}{4}" \times 5\frac{3}{4}"$	$5'-3\frac{1}{4}"$	M371
8	$1\frac{1}{4}" \times 4\frac{1}{2}"$	$11'-9\frac{1}{2}"$	M372
8	$1\frac{1}{4}" \times 4\frac{1}{2}"$	$3'-3\frac{1}{2}"$	M373
8	$1\frac{1}{4}" \times 5\frac{1}{2}"$	$11'-3\frac{1}{4}"$	M374
8	$1\frac{1}{4}" \times 5\frac{1}{2}"$	$5'-3\frac{1}{4}"$	M375
16	$1\frac{3}{4}" \times 9"$	$8'-3\frac{1}{4}"$	M376
80	$2" \times 3"$	$4'-4\frac{3}{4}"$	M239
16	$2" \times 3"$	$3'-4\frac{3}{4}"$	M240
32	$2" \times 4"$	$4'-4\frac{3}{4}"$	M241

Wall Beam #13

12	$1\frac{1}{4} \times 7\frac{3}{4}"$	$12'-2"$	M302
12	$1\frac{1}{4} \times 7\frac{3}{4}"$	$4'-2"$	M303
8	$1\frac{1}{4} \times 7\frac{3}{4}"$	$12'-3\frac{1}{4}"$	M304
8	$1\frac{1}{4} \times 7\frac{3}{4}"$	$4'-3\frac{1}{4}"$	M305
4	$1\frac{1}{4}" \times 5\frac{1}{2}"$	$12'-2"$	M306
4	$1\frac{1}{4}" \times 5\frac{1}{2}"$	$4'-2"$	M307
4	$1\frac{1}{4}" \times 5"$	$12'-3\frac{1}{4}"$	M308
4	$1\frac{1}{4}" \times 5"$	$4'-3\frac{1}{4}"$	M309
4	$1\frac{1}{4} \times 8\frac{1}{4}"$	$9'-9\frac{1}{4}"$	M310
8	$1\frac{3}{4}" \times 7"$	$9'-10\frac{1}{2}"$	M311
4	$1\frac{3}{4}" \times 7"$	$6'-8"$	M312
4	$1\frac{3}{4}" \times 7"$	$5'-5"$	M313
4	$\frac{7}{8}" \times 9\frac{1}{2}"$	$9'-10\frac{1}{2}"$	M314
4	$\frac{7}{8}" \times 9\frac{1}{2}"$	$5'-5"$	M315
8	$1\frac{1}{4}" \times 4\frac{1}{4}"$	$2'-4\frac{3}{4}"$	M316
4	$1\frac{1}{4}" \times 3\frac{3}{4}"$	$2'-4\frac{3}{4}"$	M317
4	$1\frac{1}{4}" \times 3\frac{3}{4}"$	$1'-8\frac{1}{2}"$	M318
32	$2" \times 3"$	$2'-10\frac{3}{4}"$	M214
8	$2" \times 3"$	$2'-2\frac{1}{2}"$	M228
36	$2" \times 3"$	$1'-2"$	M229
12	$3" \times 4"$	$2'-4\frac{3}{4}"$	M230
4	$3" \times 4"$	$1'-8\frac{1}{2}"$	M231
12	$2" \times 4"$	$2'-10\frac{3}{4}"$	M227
4	$2" \times 4"$	$2'-2\frac{1}{2}"$	M222

FIG. 363.—Typical material sheet for standardizing form work.

single operation. The accompanying drawing (Fig. 363) is a typical one, showing form details and lumber schedules. The form details, as previously mentioned, stated exactly the boards which would constitute a particular form and the number of forms required. Cards were made out at the route clerk's office informing the labor foreman of the number of pieces of stock and sizes to be taken to one of the mills. The mill man in turn received orders from the route clerk stating the length to cut off the stock and, if ripping was required, to what width to rip. It was the duty of the labor foreman to tag this stock according to the number indicated on his order card. When all the stock required for a certain form had been cut and delivered to the proper bench, orders were issued by the route clerk to the carpenters to make the number of forms required as indicated on the blue-print.

"In order that the forms should be made up in time to erect as planned by the office, eight benches were used with two carpenters at each bench. The carpenter having been given his orders by the route clerk telling him what lumber would be tagged to make up a certain specific form and having been given blueprint of the same form, he merely laid his stock on the bench and nailed it together. When the forms were completed a laborer oiled them and stenciled the form symbols on them. If not required immediately the forms were piled back of the bench where they were made. Since it was previously determined at which point of the building the erection was to be started, a clerk was given the duty of listing forms as required for erection and seeing that labor foreman received orders to have these forms delivered where and when required.

"According to Mr. Leonard C. Wason, president of the Aberthaw Construction Company, the labor cost on such work has been reduced 20 per cent. in the last three years as the result of detailing the forms in the drafting room and a careful study of the method of routing."

87. Steel Forms.—The application of steel forms to building construction is advancing at an exceedingly rapid rate at the present time. Steel forms are almost universally employed for circular columns and for flaring column heads. Wall forms are also much used and have given satisfaction especially in residences and other structures of the foundation-wall type.

Several types of floor forms are on the market but, as yet, have not been employed to any considerable extent. The same is true of forms for rectangular and octagonal columns.

The adjustment in height of steel forms for circular columns is obtained by telescoping the ends. Such forms are usually made up of a series of panels of thin galvanized steel held rigidly in place, like staves in a barrel, by means of stiff steel bands.

The panels are somewhat flexible and are sprung in or out depending upon the size of the column.

Steel forms are, of course, more expensive than wood in first cost but, if layouts are standardized so that the forms may be used a great many times, the steel becomes more economical than the wood if given proper attention. Thus one difficulty in using steel forms is due to the variation in design of the structural members. Another is that where steel sections are lapped, joints are apt to show badly in the finished concrete. Much attention, however, will be given to steel forms in the near future because of the increasing cost of timber.

88. Construction Notes.—All joints that are to be taken apart should have as few nails as possible. The use of clamps and wedges instead of nails is sound economy.

No attempt should be made to get cabinet work in form construction, but inaccuracies in form measurements should be guarded against.

The exact locations for columns should be laid out in a systematic manner, using the transit where necessary. The column forms should then be set true, and plumbed with an engineer's level. The beam forms usually rest on the column sides and the beam and girder bottoms should be made with a slight play so that swelling will not bind the beveled ends.

Concrete will produce an upward pressure when it is poured under horizontal or inclined forms (such as in footings) and such forms should be anchored to prevent their lifting.

One method of erecting column forms is to nail three of the sides together before setting in place, and the fourth side afterward. This enables the column reinforcement to be put in place before the form is set. Another method in common use is to put the four sides together before raising, in which case the form must be raised above the projecting reinforcement (belonging to the story below) and then lowered. Under ordinary conditions the first method seems preferable.

Whenever the widths of beams or girders change, it is convenient to place a strip of zinc over the crack that is formed in the slab sheathing. If the space is too wide, a strip of wood must be used.

Forms should usually be oiled before they are set in place, in order to prevent the concrete sticking to them and to preserve them against damage by alternate wetting and drying. Crude

oil, which is a petroleum product, is one of the best coatings, as it soaks into the wood and does not make the forms too greasy to handle. If the surface is to be plastered, the forms should not be oiled but should be thoroughly soaked with water just before concreting. Water should be used freely even when forms are oiled. Forms which are to be used again should always be cleaned as soon as they are taken down. If concrete sticks once, it is difficult to prevent it from sticking in the same place when the forms are used again.

It is advisable to raise beam and girder forms slightly higher in the center than at the ends in order to prevent sagging of the

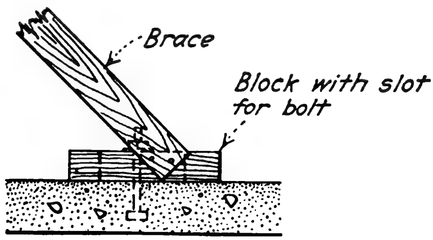


Fig. 364.

beam due to deflection and to other reasons, such as adjustment of wedges and compressing of the supports. A rough rule is to assume a deflection of the forms equal to $\frac{1}{4}$ in. in every 10 ft. of length.

Column forms may be held in position by braces nailed to adjustable blocks bolted through the concrete slab, the bolt being set in place when the slab is poured. An adjustable block of this nature is shown in Fig. 364 and the way in which such blocks are used is clearly illustrated in Figs. 342 and 357.

Posts under beams and girders are usually put in place and wedged up without any lateral bracing whatever.

The sides of beam and girder forms should lap down over the edges of the bottom plank to permit their easy removal in advance of the bottom.

In removing forms the green concrete must not be disturbed by prying against it.

Problems

30. Determine the maximum spacing of 2×6-in. joists which are to span 8 ft. and to support a 6-in. slab with a construction live load of 75 lb.

per square foot. Use $M = \frac{wl^2}{10}$ and assume the maximum allowable fiber stress in the joists and 1-in. sheathing at 1200 lb. per square inch, with deflection in each case limited to ⅛ in.

31. A wall form is filled at the rate of 5 vertical feet per hour. What is the required spacing of studs, assuming 1⅝-in. sheathing and a temperature of 70°? Use same M and same unit stresses as in Problem 30.
32. Find the maximum spacing of 4×4-in. posts to support the forms for 10×28-in. beams 7 ft. center to center with a 5½-in. slab. Use 450 lb. per square inch as the allowable stress in the posts and assume a construction load of 75 lb. per square foot.
33. Determine the maximum spacing of joists and stringers supporting forms for an 8-in. flat slab floor with a construction live load of 75 lb. per square foot. Assume 1⅝-in. sheathing and 2×8-in. joists, and assume the sheathing as fully continuous—that is, use the formula $M = \frac{wl^2}{12}$. Use the formula $M = \frac{wl^2}{10}$ for the joists, and assume a maximum allowable fiber stress in both joists and sheathing of 1200 lb. per square inch.

CHAPTER XX

BENDING AND PLACING OF REINFORCEMENT

89. Checking, Assorting and Storing Steel.—Steel should be checked, assorted, and stored as soon as it is delivered at the site. It should be blocked up several inches from the ground, and should be stored in such a manner that those rods needed first may be easily reached. A steel storage is shown in the foreground of Fig. 365 with a bench for bending the reinforcement.

90. Bending of Reinforcement.—Such a simple matter as bending of rods for concrete reinforcement might seem to be almost too unimportant a subject to be worthy of very much attention. However, when it comes to the proposition of making thousands of bends per day, factors of time and expense in this work are most important. Of course, the structural design should be such as to require the steel bends to be as few in number and kind as possible, but much can be done to lessen expense by the manner of making these bends. In addition to economical considerations, care should be taken to see that the bends are made true to line and plane, and that the steel is not injured during the operation.

In general, there are three different types of bends to be made with reinforcing rods which may be indicated as follows: (1) bending of heavy beam and girder rods, (2) bending of stirrups and column hoops, and (3) bending of slab reinforcement. In making each type of bend, the work should always be so arranged that all rods of the same size and shape are bent at the same time. This avoids remeasuring and resetting of templets.

A simple method of bending heavy rods is shown in Fig. 366. Either steel bars or steel plugs 5 or 6 in. long are placed in holes in the bending table, these holes being bored at the points where the bends are to be made. A piece of plank is nailed to the table, as shown, in order to hold the rod in place while being bent. The rod is then placed in the position GH and bent around the plugs C and D, and then around the plugs B and E until the ends AB and EF are parallel to GH.

Courtesy of Aberthaw Construction Co.

FIG. 365.—T. S. Simms Co.'s building, St. John, N. B.

The manner in which rods are to be bent is generally left to the discretion of the steel foreman. The general practice is to bend beam and girder reinforcement by using a heavy pipe slipped on the rod and then making the bends as described, using either steel plugs or angles bolted to the table.

In building two large reinforced-concrete buildings for the General Electric Co. at Schenectady, N. Y., the Stone & Webster

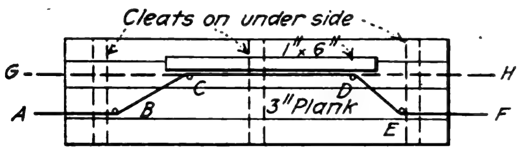


FIG. 366.

Engineering Corporation of Boston accomplished the bending of the heavy rods by means of two $\frac{3}{8}$-in. steel plates mounted one on top of the other on a bending table, and, in these, holes were drilled 3 in. on centers in both directions. Steel pegs were dropped in the holes the proper distance and angle apart, and the rods were then bent by hand using a 2-in. steam pipe.

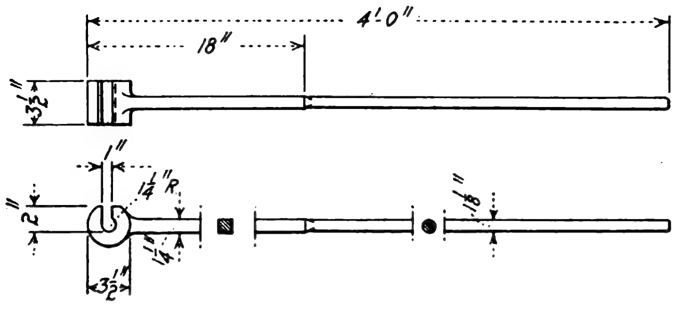


FIG. 367.

Several hand and power devices are on the market for the bending of steel reinforcement. These all have their merits and have given satisfaction.

Bending should be done in such a manner that the rods will not break or crack at the bend; that is, the bends should not be too sharp and the bending force should be applied gradually and not with a jerk. Reinforcing rods should also be bent cold except for the unusual sizes of $1\frac{1}{2}$ in. and upward, when heat may

be required. Warming up to a low cherry red should under all circumstances be the highest heat permitted. Rods to be bent are generally not larger than $1\frac{1}{4}$ in. and most hand bending machines on the market do not claim to bend rods of any greater diameter.

Some simple bending apparatus is usually arranged for bending column hoops and beam stirrups of the ordinary type. Some steel men, however, bend their stirrups in practically the same manner as the heavier rods.

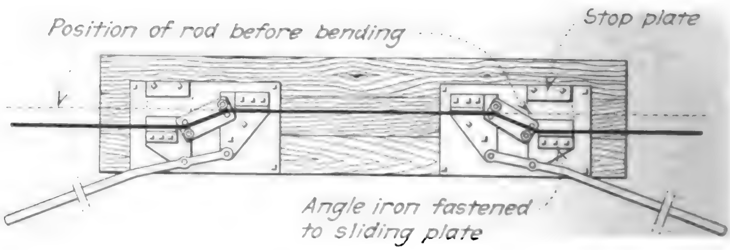


FIG. 368.

Slab reinforcement is usually bent after it is in place on the floor. A tool for this purpose is shown in Fig. 367. Sometimes, however, slab rods are bent on a bending table by means of a specially-designed rod bender. This latter method seems preferable since the bends can be more accurately made. To keep the cost of such bending as low as possible, the machine should be constructed so that the bends may be made with great rapidity. Such a machine is shown in Fig. 368 and was employed on the General Electric Co's. buildings referred to above.

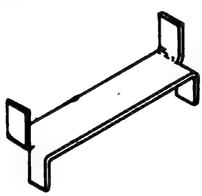
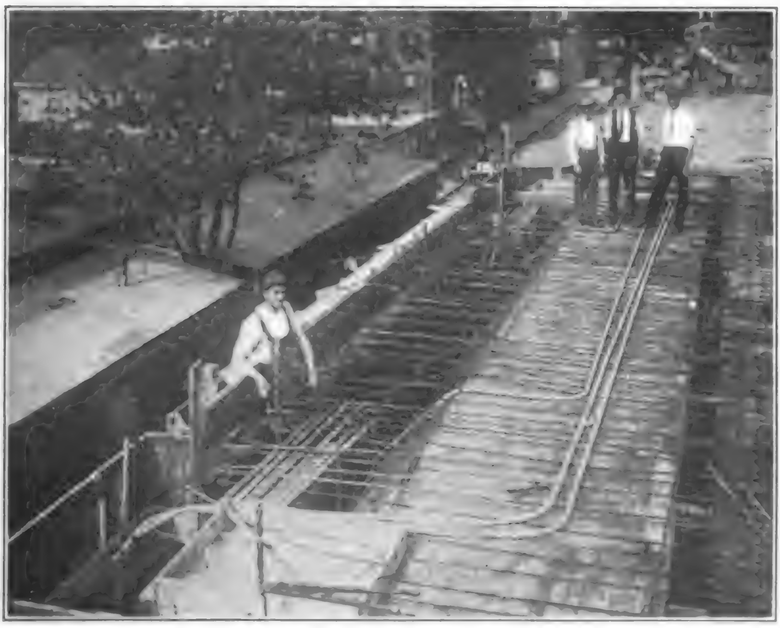


FIG. 369.

This machine consists of two units which can be placed any distance apart and securely bolted down to a table. Each part consists of a steel plate on which is mounted two smaller plates having angle stops attached. Between the stops is located a casting, which is pivoted so that it can readily take any desired position between the angle stops. With the steel rod in position, this casting can be turned by means of a long lever attached so as to move one of the mounted plates. On the upper side of the casting are four lugs, and two of these lugs are constructed so that adjustable collars can be slipped over them, thus making a

Courtesy of Turner Construction Co.

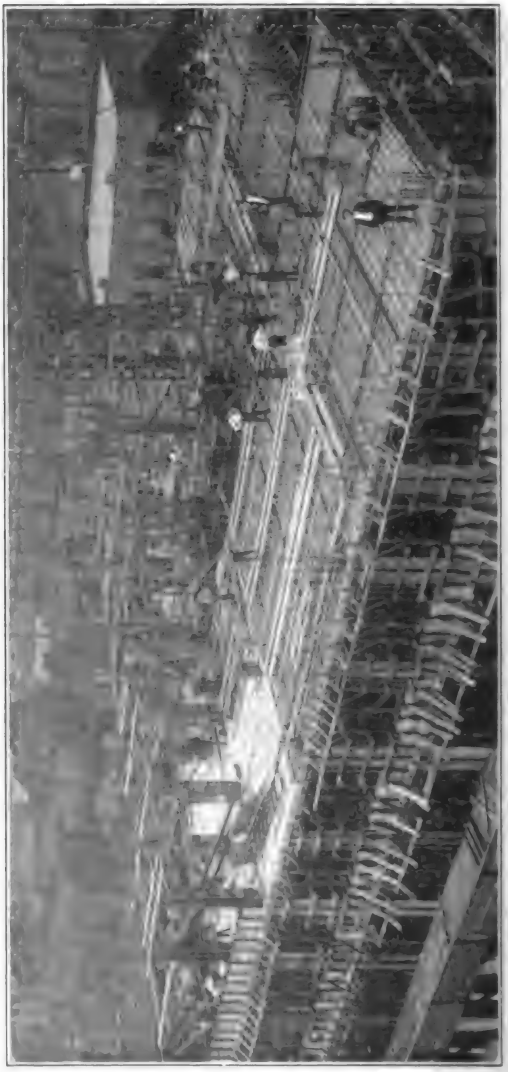
FIG. 370.—Phelps Publishing Co'.s building, Springfield, Mass.



Courtesy of Turner Construction Co.

FIG. 371.—Same building shown in Fig. 370. Reinforcement in place for girder over entrance. Note that the center wall column carrying six stories comes in the middle of the girder.

tight fit for the rods which are to be bent. The amount and the angle of the offset can be regulated by changing the distance through which the lever is turned. This machine offsets a rod



Courtesy of Ferro-concrete Construction Co.

FIG. 372.—Empire American Bank building, Seattle, Washington.

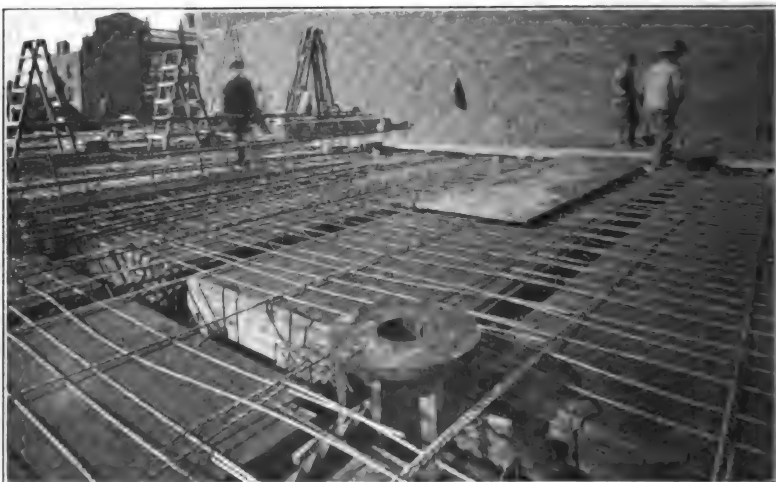
parallel to itself and with one pull of a lever two bends can be made.

91. Placing of Reinforcement.—Steel should be thoroughly

Courtesy of Aberthaw Construction Co.

FIG. 373.—Steel reinforcement. T. S. Simms Co.'s building, St. John, N. B.

cleaned before being placed in the forms in order to obtain a positive adhesion of the concrete to the steel. A slight film of red rust is not objectionable, but no rod should be set in place on which rust scales have formed. (See Art. 24, Volume I.) The advantages to be derived from placing reinforcement in frames has been considered in Art. 17. With steel in frames the erector has simply to line and level them in the forms, place braces where necessary, and make end connections with abutting frames. Spacing chairs are usually employed with slab, beam, and girder reinforcement to hold the bottom rods up from the form so that little bracing or none is required. The chairs are sometimes blocks



Courtesy of Turner Construction Co.

Fig. 374.—Eastman Kodak office building, West 23d Street, New York City. Reinforced-concrete floors supported on cast-iron columns. Note wall-column reinforcement made up and laid horizontally on the floor at the left of the picture.

of concrete with grooves in the top; or sometimes a thin square steel plate about 4 in. on a side with a hole in its center, the halves of which are bent to an angle of 60 to 90 degrees with each other to form feet for the chair, the hole forming a seat for the reinforcing rod. Another type of chair is shown in Fig. 369.

Wall and column reinforcement should be made up into frames, the same as for beams and girders. In some cases, it would be best to assemble even slab reinforcement on the ground and place it in assembled units.

Courtesy of Aberthaw Construction Co.

FIG. 375.—Steel reinforcement. Cartridge storage building, Winchester Repeating Arms Co., New Haven, Conn.

Courtesy of Ferro-concrete Construction Co.

FIG. 376.—Kraus building, Memphis, Tenn.

Courtesy of Ferro-concrete Construction Co.

FIG. 377.—Continental Motor Car Co.'s building, Detroit, Mich.

All reinforcing metal should be put in place in the form and securely fastened in correct positions by wiring or otherwise before the placing of the concrete is begun. Particular attention should be given to loose-bar reinforcement—that it is accurately and properly supported in position and that it is not disturbed until the concrete is poured. Steel in position is shown in Figs. 68, 141, 159, and 371 to 377, inclusive.

CHAPTER XXI

PROPORTIONING, MIXING, AND PLACING OF CONCRETE

92. Proportioning of Concrete.—The following methods of proportioning concrete are in use:

(1) Arbitrary selection of proportions.
(2) Void determination.
(3) Proportioning by trial mixtures.
(4) Proportioning by mechanical analysis.

In discussing methods (3) and (4), dry aggregates only will be considered in the weight determinations. If an appreciable amount of moisture is present in the aggregates, this should be determined by drying the sand and stone in an oven until there is no further loss of weight.

Proportioning by Arbitrary Selection.—The common rule-of-thumb method of proportioning concrete may cause wide discrepancies because of the variation in the character of the ingredients. In fact, arbitrary selection should never be permitted except where the work is not expensive or important enough to warrant special proportioning of the materials and grading of the aggregates.

If arbitrary proportioning is decided upon, the best method to follow is to proportion the cement to sand by judgment in accordance with the character of the construction, to adopt (as a trial) twice as much stone as sand by volume, and then to vary this proportion using as much of the coarse aggregate as possible without producing noticeable voids or stone pockets in the concrete. In cases where the coarse material contains a great many small particles, the proportion of sand should be tried at somewhat less than one-half the volume of stone.

Experience in the handling of concrete enables one to judge readily whether the mortar is deficient or not. With too little sand, stone pockets are apt to occur on the surfaces, and it is difficult to fill all the voids in the stone; with too much sand, a harsh working of the concrete will be noticed or an excess of mortar will rise to the top when placing.

Proportioning by Void Determination.—There are four distinct methods of proportioning concrete by the determination of voids:

(a) Finding the voids in the stone and in the sand, and then proportioning the materials so that the volume of sand is equivalent to the volume of voids in the stone and so that the volume of cement is slightly in excess of the voids in the sand. (The excess of cement is provided on account of inaccuracies in the void method of proportioning.)

(b) Finding the voids in the stone, and, after selecting the proportions of cement to sand by test or by judgment, proportioning the mortar to the stone so that the volume of mortar is slightly in excess of the voids in the stone.

(c) Mixing the sand and stone and providing such a proportion of cement that the paste will slightly more than fill the voids in the mixed aggregate.

(d) Making trial mixtures of dry materials in different proportions to determine the mixture giving the smallest percentage of voids, and then adding an arbitrary percentage of cement, or one based on the voids in the mixed aggregate.

Proportioning by the determination of voids is little (if any) better than the proportioning by arbitrary assignment. In the first place, the percentage of voids in sand is greatly affected by even a small percentage of moisture. When sand is moistened, a film of water coats each particle of sand and separates it by surface tension from the grains surrounding it. Fine sand, having a larger number of grains, and consequently, more surface area, is more increased in bulk by the addition of water than coarse sand. Again, the volume of water required in mixing actually occupies space in the resulting concrete and not only every cement but also every sand has a different and definite percentage of water necessary to bring it to what may be called normal consistency. Voids in absolutely dry sand are certainly no criterion of its qualities for concrete, while a moist sand will give different results on different days.

Inaccuracy in proportioning by void determination is also due in part to the difference in the compactness of the materials under varied methods of handling, and to the fact that many grains of the sand are too coarse to enter the voids of the stone and consequently thrust apart the particles of the large aggregate. Many of the voids in the sand are also too small for the grains of cement to fit into them without expanding the volume.

The methods of determining voids in sand are explained in Art. 79. Voids in stone may be found by direct water measurement.

Proportioning by Trial Mixtures.—The fact that the densest concrete is the strongest and most impermeable (Art. 7, Volume I) makes possible a convenient and accurate method of proportioning concrete and of determining the relative value of different aggregates. Mr. William B. Fuller describes the method as follows:

"Procure a piece of steel pipe 8 to 12 in. in diameter and a foot long and close off one end; also obtain an accurate weighing scale. Weigh out any proportions selected at random, of cement, sand, and stone, and of such quantity as will fill the pipe about three-quarters full, and mix thoroughly with water on an impervious platform, such as a sheet of iron; then, standing the pipe on end, put all the concrete in the pipe, tamping it thoroughly, and when all is in, measure and record the depth of the concrete in the pipe. Now throw this concrete away, clean the pipe and tools and make up another batch with the total weight of cement, sand, and stone the same as before but with the proportions of the sand to the stone slightly different. Mix and place as before and measure and record the depth in the pipe, and if the depth in the pipe is less and the concrete still looks nice and works well, this is a better mixture than the first. Continue trying in this way until the proportion has been found which will give the least depth in the pipe. This simply shows that the same amount of materials is being compacted into a smaller space and that, consequently, the concrete is more dense. Of course, exactly similar materials must be used as are to be used on the work, and after having in this way decided on the proportions to be used on the work it is desirable to make such trials several times while the work is in progress, to be sure there is no great change in materials, or, if there is any change, to determine the corresponding change in the proportions.

"The above-described method of obtaining proportions does not take very much time, is not difficult, and a little trouble taken in this way will often be productive of very important results over the guess method of deciding proportions so universally prevalent.

"A person interested in this method of proportioning will find on trial that other sands and stones available in the vicinity will give other depths in the pipe, and it is probable that by looking around and obtaining the best available materials the strength of the concrete obtainable will be very materially increased."

Proportioning by Mechanical Analysis.—The most scientific method of proportioning concrete is by mechanical analysis.

This method is explained at length in Art. 8 of Volume I, to which the reader is referred.

Mechanical analysis affords a means of determining the best proportions in which to mix the fine and the coarse aggregate, and shows how the aggregate may be improved by adding or subtracting some particular size. This method should be employed on all large or important structures or wherever the expense of screening and proportioning is likely to be justified by the better quality, or the less cost, of the concrete secured. The analysis should be made regularly to determine whether or not the sizes of the aggregate have changed, and, whenever a change is detected, the proportions should be varied to secure the most economical and densest concrete.

An article written by Mr. L. C. Wason, President of the Aberthaw Construction Co., and published in the *Concrete Engineering*, May, 1909, is of value and is quoted in part as follows:

"In the work of the Little Falls Water Co's. filtration plant, where over 8000 yd. were placed, bins were used to size the material into five different grades. The engineer made careful analysis of the material and combined these in such a way as to obtain waterproof concrete mixed in the proportion approximating 1 part cement, 3 parts sand, and 7 of stone. The actual saving in cement more than paid for the extra cost and trouble of sizing and testing the aggregate. This condition is not often possible, and therefore the ordinary precaution is that of using a little richer mixture than ideal conditions would require. This is the most usual and common factor of safety. On large and important work where speed as well as accurate results are required, the writer has erected elevated bins, fed material into them with an elevator which discharged into a rotary screen, sizing the material into three or more grades. From these bins material was drawn into accurate measuring hoppers and dumped directly into the mixer, thus speed, accuracy, and uniformity of results were obtained. The expense of this mechanical plant is such that unless 1500 or more yards of concrete are to be mixed, it is not advisable to attempt it.

"For small work industrial cars, wheelbrarows, or measuring frames are a necessity, and, in connection with measuring frames, the greatest inaccuracies are obtained in the usual shape used. Assume a 1: 2: 4 mixture which makes nearly 16 cu. ft. of concrete. This is ordinarily measured in a frame made of 8-in. plank, enclosing an area of 4 ft. by 6 ft. If, as is common, the frame is laid on small stones left on the platform from a previous batch, increasing the depth by ½ in., an extra cubic foot of stone goes into the mix, whereas if a frame 3 ft. by

2 ft. 8 in., 2. ft deep, is used, the same volume is obtained and then an error of $\frac{1}{2}$ in. in the depth would amount to only $\frac{1}{3}$ ft.

"When broken stone is used, it has been the writer's experience that there has been very little variation in the voids, so that seldom does any change have to be made in the mix as the work progresses. Throughout New England the stone is always sized with a rotary screen before its delivery. The simplest and most effective way of testing if the concrete is densely proportioned for ordinary work (where the refinement necessary in waterproofing is not the essential feature) is, after the batch is mixed by hand or machine and is lying loose ready to be placed, to tap the surface lightly with the sole of the foot three times. If the surface becomes smooth, the mortar just filling the voids of the stone, the concrete is all right. If the surface is porous, there is not enough mortar, and vice versa. A little practice and observation of the concrete in place will soon produce a trained judgment.

"In this vicinity there are large quantities of good gravel available, and this is frequently used in mixing concrete. The writer has never seen a gravel pit where the materials were rightly proportioned and where the run of the pit could be used without change. There is always an excess of sand. If the run of the pit is taken, it should be tested frequently to find the actual proportion of sand and stone and then enough extra stone added to obtain the proper mixture, and as the same pit varies constantly very frequent tests should be made. It is customary, because it is almost always necessary, to screen the sand from the stone and then to recombine them in the correct proportions, and when this is done the same method of measuring and testing the result can be used as with broken stone, slag, or other form of aggregate."

93. Mixing of Concrete.—All concrete for reinforced work should be mixed by machine. Although with careful superintendence hand-mixing will give first-class results, machine-mixed concrete is usually of more uniform quality than that mixed by hand, and is generally less expensive—except, of course, where the quantity of concrete is so small as to prohibit the expense of purchasing or renting a mixer. The engineer should preferably reserve the right to permit hand-mixing if practically unavoidable, but this method of mixing should be resorted to only when machinery is unobtainable or where it is necessary to start work on a large job before the machinery has arrived.

The proper degree of mixing may be determined simply by the appearance. It is an indication that the operation has continued long enough when the mass assumes a uniform color and a dense, smooth appearance. Some contractors mix the materials dry

until a uniform color is secured and then add the water. Others put in the material and the water at once. Either way will produce good results, except for hand-mixing, where the mixing of the cement and the sand in the dry state is the general practice.

The strength of concrete is very largely dependent upon the thoroughness of mixing, and much care is needed in this part of the work. No matter how suitable for the purpose the materials and proportions of the same may be, insufficient mixing will result in inferior concrete.

The greatest care should also be exercised to make sure that the specified amounts of the materials go into each batch of concrete. For measuring concrete aggregates, it is not good practice to use the common form of contractor's wheelbarrow because the loads vary considerably with the variation in the heaping of the barrow. Special barrows constructed with sides nearly vertical can be obtained which will give the required amount when level full.

The amount of water used in mixing is largely a matter of judgement. Sufficient water should be added to make the mixed concrete into a flowing paste that will flow readily around the reinforcing steel and requires only light tamping or puddling to bring the mass to a homogeneous condition. A slight excess of water is preferable to not enough, but there should not be any appreciable quantity of free water present to wash the cement off the stone. Concrete is mixed with an excess of water if pools are immediately formed on top of the concrete when deposited in the forms. Although the quantity of water needed in different batches will vary occasionally because of the condition of the materials, the amount to use can be regulated best by measurement. A tank with a float fastened to an indicator on the outside is easily constructed in connection with a concrete mixer.

Concrete mixing machines may be divided into two general classes: (1) continuous mixers, into which the materials are fed constantly and from which the concrete is discharged in a steady stream, and (2) batch mixers, into which sufficient materials are placed at one time to make a convenient-sized batch of concrete, the whole amount being discharged in one mass after it is mixed. Many machines are adapted to either continuous or batch mixing. Good results can be produced by either batch or continuous mixing but the correct proportions are more readily

secured by mixing the concrete in batches, and this method is generally preferred. Many specifications prohibit the use of continuous mixers for reinforced-concrete work.

The order adopted for mixing concrete materials by hand has little effect upon the strength of the concrete, provided the materials are turned a sufficient number of times to insure a perfect and equal distribution of the cement through the entire mass. The usual method is to mix the cement and sand dry, shovel the mixture on to the stone, add the proper amount of water, and then turn the mass three times.

94. Transporting and Placing of Concrete.—The method to be employed in conveying concrete from the mixing place to the work depends to a great extent on local conditions. Wheelbarrows, two-wheel hand carts, cars on tracks, derricks, hoists, cableways, and gravity chutes are variously used.

The ordinary contractor's wheelbarrow served fairly well for conveying concrete during the time that dry concrete was considered good practice, because it could be heaped. At the present day, however, it is practically discarded for this purpose and has given way to special 2-wheel concrete barrows or carts which hold as much concrete as a man can readily handle. Such carts require wide runs or platforms (Figs. 376 and 377) and cannot be pushed up an incline. Also, turnouts should be provided occasionally to avoid delay when an empty barrow meets a full one. Where carts are run over floor forms with slab steel in place, the runs of plank should be spiked to substantial cleats so as to raise the planking above the forms and carry it over the slab steel.

Small dump cars on tracks are frequently used where the concrete is deposited at some distance from the mixing plant. These cars run on light rails placed on plank runs, as shown in Figs. 159 and 373. A turnout is shown in the first figure mentioned. The runs are supported on timber horses so constructed that they may be easily taken out of the soft concrete.

Hoists are generally employed in building construction to convey the concrete to the desired floor level. The concrete may be lifted either in barrows placed on an elevator or in a regular hoist bucket. The latter method is usually less expensive. Especially is this true when a hopper is used to hold the concrete instead of dumping from the hoist bucket directly into the barrows. A hopper should hold at least a batch and a half of con-

crete in order not to restrict the output of the mixer. A hopper is clearly shown in Fig. 332.

A successful way of distributing concrete from a hopper to any part of the work by gravity is shown in Figs. 340, 341, and 375. The hoist bucket dumps into a hopper at the required elevation, from which it flows to a trough, or else to a galvanized iron pipe, which is operated by cables, booms, and swivels. The following general instructions for operating a spouting system, issued by the Chain Belt Co., Milwaukee, may be of value:

1. Run about 10 ft. of water in skip.
2. Charge mixers, using mostly sand with the cement and plenty of water.
3. Run up skip with water and drop at once, following up as quickly as possible with a charge of concrete.
4. Run the mixer so that it may have the consistency of a thick gravy, so that when it levels off the rock is seen held in suspension. If, when dumped in the skip, it stands up, it is too thick; if it levels off and shows 1 in. of water, it is too thin.
5. There should be a man in the tower to operate the concrete gate on the hopper to regulate the flow of concrete through the pipes. The stream should fill about one-third of the conveyor pipe and the charge should be so timed that the stream will be as nearly continuous as possible.
6. Continuous and successful running depend upon the uniform mixture. Be careful not to get it too thin. After a shut down of over 10 min., flush the pipe by sending up 10 ft. of water in the skip.
7. At the end of the day's run, clean up the mixers with two charges of water. Send these through the pipes.
8. All pipe joints should be oiled with thick grease, so as to prevent sticking.

Before starting to place concrete it is very important to see that the forms are plumb and properly braced; that all dirt, shavings, sawdust, and other rubbish have been removed; that the steel reinforcement is in its proper position and securely wired; and that the forms have been either thoroughly oiled or wet down with a hose, or both, as the case may be. If the concrete hardens in the hoppers or in the wheelbarrows, it should not be used, since retempering such concrete with more water is dangerous; under such conditions the concrete hardens very slowly and probably never reaches the calculated strength. Concrete

which stiffens perceptibly in the receiving hopper or barrows should never be used.

Concrete may be handled and placed in any way best suited to the nature of the work provided the materials are not allowed to separate, and provided also that the time of mixing and pouring a batch of concrete is not greater than the time-set of the cement. Concrete should be so deposited that it will not flow along the forms. It should either be dumped slowly, or shoveled out of receptacles, and the dumping should not be from great heights. If allowed to slide down a metal tube or enclosed trough, the tube should be kept continuously full. Excess of water is another common cause of separation and must be avoided.

Wet concrete such as is used in reinforced-concrete construction needs no ramming. It is sufficient to spade or thoroughly stir (puddle) the concrete by means of forks, spades, or other suitable instruments. This is done in order to force the entrained air and surplus water to the surface, and thus prevent pockets or voids. Under no circumstances should aggregate free from mortar be allowed to gather around the reinforcing rods and all stones should be carefully but judiciously spaded away from concrete faces. Faces exposed to view may be made smooth by thrusting a square-pointed spade or a thin tool through the concrete close to the form to force back the larger stones and prevent stone pockets. Gentle tapping of forms also helps to give smooth surfaces by shaking down and consolidating the concrete and forcing out air bubbles.

Reinforcing rods should not be jarred after concrete begins to set, for obvious reasons. Also no walking should be allowed on green concrete for at least 12 hours after it is poured, as otherwise its strength may be seriously impaired.

It may be necessary to cover green concrete against the direct rays of the sun in hot weather. Whether or not this is done, the concrete should be sprinkled liberally at frequent intervals for the first two or three weeks to make up for the loss of water by evaporation and permit it to set properly. Keeping concrete wet also prevents to a large degree contraction in hardening and the setting up of initial stresses. Concrete may be kept continuously wet by employing either wet sand, sawdust, or burlap as a covering.

In monolithic building construction, the columns should be poured at least 6 to 8 hours ahead of the beams and girders to

allow the concrete in the columns to settle and shrink, and the operation should be continuous from the base to the underside of the supported beam or girder. In like manner, the slab and the beams and girders are usually run in one continuous operation, but where T-beam action is not considered the slab may be poured separately.

Joints in concrete work should not occur except at the end of a day's work. Such joints should be straight, well-defined lines, where they will do the least harm in case of poor bond with the next day's concrete. Where work on beams and slabs must be suspended, the joint should preferably be made through the center of the span of all slabs and beams as the concrete at this section is in compression, the tensile stress is all taken by the reinforcement, and the shear at this point in most beams is zero. Joints in concrete should always be at right angles to the line of thrust and should not be where there is any considerable shear.

Bonding new to old concrete is a matter that requires the greatest care. The surface should be thoroughly cleaned of all foreign material and laitance or scum, and then rubbed with stiff wire or willow brooms, removing any loose stones at the surface. The surface should finally be washed with water from a hose to remove all loose chips and dust. The secret of securing a good bond lies largely in getting rid of the glazed skin and the slime and dust which forms on the surface of concrete that has set. Live steam at a high pressure has been used to clean concrete surfaces and has given complete satisfaction. The steam should be applied after the surface of the old concrete has been brushed as clean as possible with stiff brushes. It is important that old concrete be well soaked with water before new concrete is joined to it, and the bond may be improved in many cases by rubbing the surface with neat cement paste or with a rich cement mortar before concreting. Care should be taken that the paste is not allowed to dry out or set before the new concrete is placed.

95. Concreting in Freezing Weather.—If proper precautions are taken, the results obtained by placing concrete in winter are practically as satisfactory as those secured in summer weather. The cost of winter concrete work, however, is generally from 6 to 10 per cent greater than in summer, but this is usually offset by the earlier occupancy of the building.

Since water is the element which brings about the hardening of concrete, it is evident that it must be kept in a fluid condition

and not allowed to freeze. Although authorities differ as to the ultimate effect of freezing upon Portland cement concrete, it is now generally admitted that freezing will not injure concrete which has had a chance to harden under favorable conditions for about 48 hours. If concrete is frozen before it takes its initial set there will be no permanent injury if upon thawing it is not again frozen until it has had an opportunity to harden. Alternate freezing and thawing at short intervals seriously affects the strength of concrete.

When concreting in winter, one or more of the following methods may be employed to prevent concrete from freezing:

(1) Lowering the freezing point of the mixing water by adding substances such as salt or calcium chloride.

(2) Heating the sand, stone, and water.

(3) Protecting the top or exposed surface by covering with panels of light sheathing, tarpaulins, building paper, or some other suitable material.

(4) Enclosing or housing the work under construction.

(5) Artificial heating of the enclosed space.

The particular method or methods to be employed will depend largely upon the character of the work—mass work not requiring the same care and protection as thin walls, columns, beams, and floor slabs.

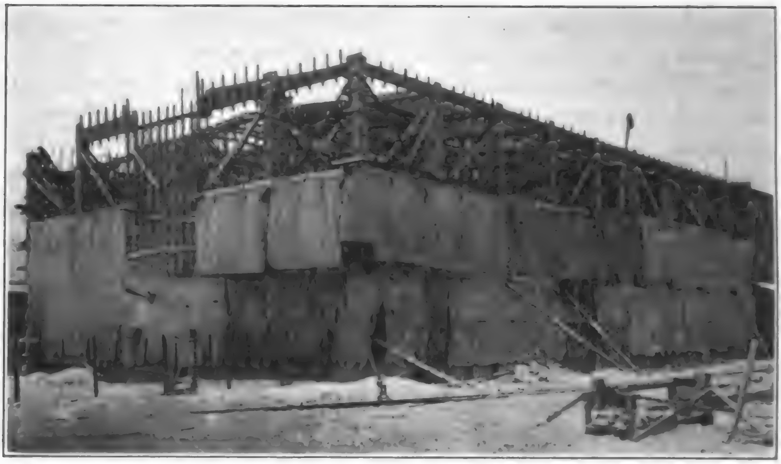
Lowering the freezing point of water by the addition of ordinary salt is probably the simplest and cheapest of the methods above mentioned but salt retards the setting and, if used to excess, will affect the ultimate strength. Approximately 1 per cent by weight of salt to the weight of water is required for each degree below 32 °F., but more than 10 per cent of salt is not considered safe. Salt, therefore, is not effective for temperatures lower than 22° F. The convenient way to add the salt is by putting it into the mixing water. Calcium chloride in quantities not over 2 per cent of the weight of the cement lowers the freezing point in an effective manner to a value between 14° and −2° F., but a larger amount hastens the set to such an extent that it is difficult to handle. Tests tend to show that salt and calcium chloride cause a corrosive action on reinforcing steel, but experiments are needed to determine whether or not such action would continue until the effective strength of the steel would be dangerously impaired.

The most satisfactory results of winter concrete work have been

obtained where the aggregates and the water are heated and the concrete work protected until it has obtained sufficient strength to withstand the action of frost. Heating the materials accelerates the rate of hardening and lengthens the time before the mixture becomes cold enough to freeze. Results show, however, that the materials should not be heated much over 90° F. to 100° F., as the concrete is weakened if the set is too rapid.

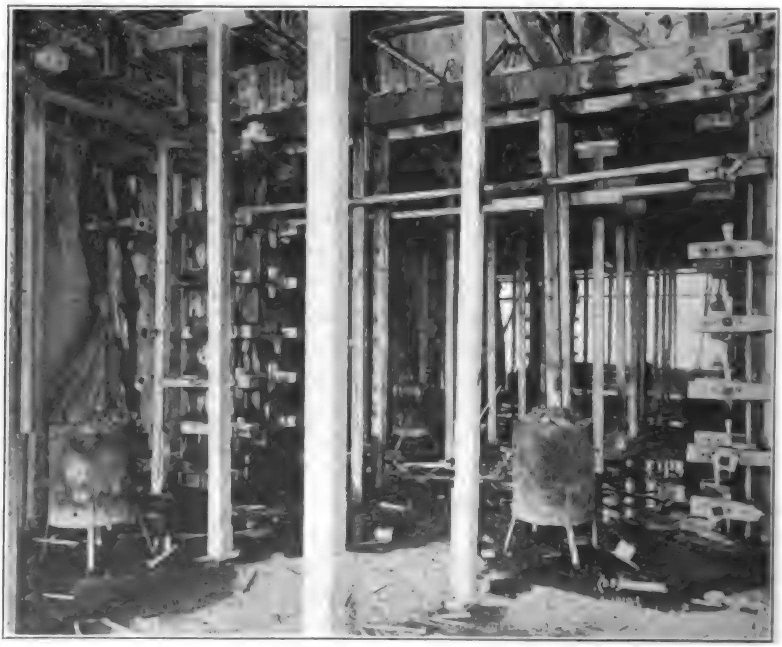
Many methods are used to heat the materials but, in any case, the additional equipment required is very simple. The best method of heating the water is to provide a suitable tank, and heat the water therein by a coil of pipe supplied with steam from the boiler running the mixer engine. The water barrel at the mixer should not be used for this purpose since in most cases the water in the same is used too fast to permit of its being properly heated. The method employed in heating sand and stone depends upon whether these materials are received by rail or wagons. Where material is received on railroad cars, it is good practice to run a steam pipe alongside the tracks with tees at frequent intervals from which steam hose is taken directly into the car itself. For heating sand perforated steam pipes are usually run through the pile and the pile kept covered with canvas. Half cylinders of sheet steel set on the ground in the form of an arch have been used successfully for heating sand and stone. Also metal culvert pipe and sections of old boilers have given satisfactory results, the fire being placed inside. Steam heating, however, is considered better than using heaters, inasmuch as the material next to the heating surface in the latter method may become damaged.

In building construction the general method of procedure is to enclose the building with canvas (Fig. 378), floor by floor, until the concrete is set, and place salamanders burning coke (Fig. 379) directly underneath all freshly poured concrete. These salamanders should be kept burning continuously until the concrete is thoroughly set. The tops of freshly poured floors are often covered with sawdust or with paper and about 12 in. of hay or straw, the latter being sometimes sprinkled with salt. In no case should fresh manure be placed over very green concrete to protect it from freezing, as it will spoil the surface of the concrete. Before placing the concrete, all forms should be swept clean of snow and all ice present should be removed with a steam jet. The mixing plant should be close to the place of concreting so as not to give the concrete a chance to freeze while it is being



Courtesy of Universal Portland Cement Co.

FIG. 378.—Concreting in winter. Wabash Screen Door Co.'s building, Minneapolis, Minn.



Courtesy of Turner Construction Co.

FIG. 379.—Factory of John F. Galvin, Long Island City. Interior during winter weather construction. Note salamanders burning to keep temperature above freezing-point. Also, note column forms.

transported. The chuting of concrete in freezing weather should not be allowed.

In the issue of *Engineering News*, May 23, 1912, Mr. F. E. Schilling, Superintendent of Construction for the Turner Construction Co. of New York City, describes winter construction on a seven-story reinforced-concrete building for the Barnet Leather Co., Little Falls, N.Y. His description is in part as follows:

"The following plant was installed in order to keep the concrete from freezing, when cold weather set h.p. An 18 h.p. boiler was rented and installed close to the building, as shown in Fig. 380. Three lines of 1½-in. steam pipes were run from the boiler; the first line leading to the water barrel at the mixer, the second leading up to the floor which

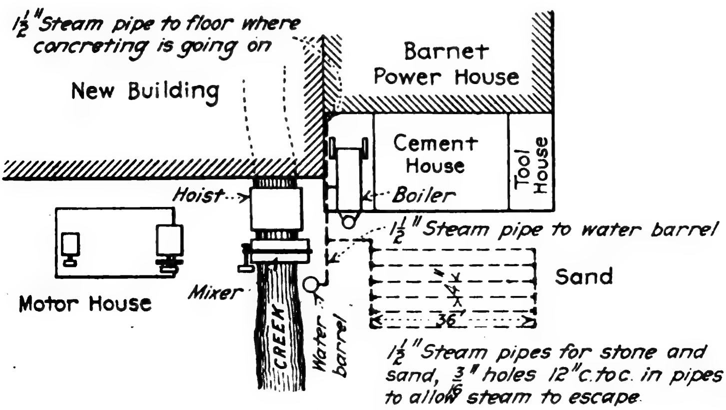


Fig. 380.

was to be concreted, and the third line leading to a coil of pipes used for heating the sand and stone. This coil consisted of six lines of 1½-in. pipes, 36 ft. long, placed 4 ft. apart and all connected at the ends. All pipes were provided with $\frac{3}{16}$-in. holes, spaced 12 in. apart.

"The sand and stone brought in by 2-cu. yd. bottom-dump wagons were piled directly on this coil, the area being enough to store 80 cu. yd. of sand and 150 cu. yd. of stone. The steam, when turned on under the sand and stone, escaped through the holes in the pipes and thawed out the material above. During very cold weather the sand and stone were covered over with heavy canvas and with steam turned on underneath, it was found that the material within ten hours was in good shape, the temperature ranging from 60 to 70° F. This was true even in the coldest weather when the sand arrived in large frosted lumps. With a steam pressure of from 25 to 35 lb. at the boiler, the

water in the barrel at the mixer would heat up to a temperature of 85° within a few minutes, and was kept at an even temperature by means of a valve on the steam line.

"The steam was carried about on the floor forms by means of 1½-in. hose. In this way all the snow and ice on the forms or on the steel was melted. When concrete was poured on the floors, this hose was kept going at full pressure all day. The hose was lowered into each column form before pouring in the concrete, and kept there until the preceding column was filled. While filling the floor slab, the hose was kept in the girder or beam directly ahead of where the concrete was poured, and the steam instead of immediately rising would follow along the beam or girder, thus warming up the steel. The concrete when dumped from the carryalls was at a temperature of 50°, so any slight difference in temperature between the steel and concrete would adjust itself when the steel was surrounded and would not lower the temperature of the concrete enough to make it freeze.

"In order to keep the concrete from freezing after it was placed in the forms, it was necessary to keep heat around it until it had set. To accomplish this, the contractor purchased enough canvas in pieces 16 ft. square to cover two stories around the sides, and also to cover over one floor after it was concreted. The carpenters, while building their exterior beam forms, nailed a 2×4-in. timber, 10 in. outside of the forms, and the canvas was hung from this timber. This allowed a space of 10 in. outside of the form for heat circulation. Coke burned in salamanders was used to keep the inclosed stories warm. The salamanders were distributed in the proportion of one to each 300 sq. ft. of floor, and at this rate, the temperature under the floor forms was kept between 80 to 90°. The hardest place to heat was found to be the bottoms of the exterior columns and, to offset this, two stories instead of one were inclosed with canvas. When the story below the one being filled was inclosed and heated, the contractor found no difficulty in keeping all the columns from freezing.

"Small boxes, 4 in. deeper than the thickness of the concrete on the floors, were placed on the floor forms before concrete was poured, and a hole was cut in the form underneath each box. As a portion of the floor was finished up, long 4×4-in. timbers were placed from box to box, old panels and plank were placed to span between the timbers, and the whole area covered with canvas. This left a 4-in. air space between the top of the concrete and the canvas and the heat rising through the holes cut in the forms under the boxes kept this space warm. The specifications for this building called for the finish on the floors to be put in after the windows and walls had been placed, so the concrete was left rough and caused no trouble in covering up.

"The coldest day that concrete was placed on any of the floors was Jan. 26, 1912, when the sixth-story columns and the seventh floor

were filled. On this day, the temperature at 7 a. m. at time of starting work, was 6° below zero. At 10 a. m., the temperature was zero; at noon, 10° above; at 3 p.m., 12° above; and at 5 p.m., 8° above. All of the concrete placed on this day turned out in good condition. Coke was kept burning in the salamanders under the forms for all floors for at least six days after the concrete was placed. No shores or forms were removed during this time.

"In order, therefore, to make speed and keep the men working, two sets of forms were necessary. Usually on the day following the concreting of a floor, no work was attempted on the next story. It was found, however, that by allowing two nights and a day heating, the concrete was hard enough on top to work on, so the covering over the top was removed and work was started on the next story. Thus while the second set of forms were being set up above, the forms still remained under the floor below, which was inclosed and heated. In this way, it was only necessary to stop work one day for every floor."

CHAPTER XXII

FINISHING CONCRETE SURFACES

96. Methods in General.—The treatment of concrete surfaces should be determined almost wholly by the character of the structure. If appearance is not an important consideration and if the forms are tight, a suitable surface may be obtained simply by careful spading, making sure that the mortar flushes to the face. If, on the other hand, the character of a building and its location are such as to warrant a finish somewhat superior to that obtained by spading, then one or more of the following methods may be used: (1) rubbing the surface with a block of carborundum or similar substance after the forms are removed, (2) brushing, (3) tooling, (4) sand blasting, (5) using aggregates of the proper size and color and then properly finishing the surface, (6) using coloring matter in the facing mixture, and (7) applying specially prepared paints or cement stains to the concrete surface.

Plastering is still another method of surface treatment but is unsatisfactory in climates where the temperature in the winter months falls below freezing. If the form work is made very rough, fair results may be obtained by this method since plaster clings best to a rough surface; plastering, however, is an art more than a science so that skilled labor is undoubtedly a most essential feature. Plastering should especially be avoided on outside work, as it is practically impossible to apply mortar in thin layers and make it adhere for any length of time. This is due, no doubt, to the shrinkage of the mortar after the shrinkage in the concrete itself has practically ceased. The richer the mortar, the more it shrinks, and the shrinkage results in leaving a hair plane between the plaster and the concrete work. The frost does the rest.

97. Rubbed Finish.—If a rubbed surface finish is desired, the coarse aggregate should be well spaded back from the face of the work and the forms should be removed before the concrete has set hard, preferably in a day or two after the concrete is poured. The process of rubbing consists in grinding down the surface of the concrete sufficiently to remove all impressions of the timber

or other irregularities, using a brick of carborundum, emery, concrete, or soft natural stone. In connection with the rubbing (which is accomplished with a circular motion), a thin grout composed of cement and sand should be applied to the surface, well rubbed in, and the work afterward washed down with clean water. The grout is used simply to fill surface imperfections and care must be taken not to allow it to remain as a film on the surface.

This method of treatment produces a comparatively smooth surface of uniform color much superior to that obtained by the all too prevalent method of painting with a grout, which almost invariably crazes, cracks, and peels off. Rubbing is a very acceptable, cheap way of finishing concrete surfaces.

98. Brushed Finish.—A brushed finish is obtained by removing with a brush the surface film of mortor which forms over the aggregate. This should be done while the concrete surface is still green. The brushing should be started just as soon as it is possible to do so without removing particles of aggregate. The time required for sufficient hardening can only be determined by experimenting with the particular surface. The forms should be constructed so they may be taken down in sections and give access to the face of the concrete without taking away the standards which provide strength for the forms as a whole.

The process consists of scrubbing or brushing the concrete surfaces with ordinary fiber (or wire) brushes and water until all the surface cement has been washed off, leaving the coarse aggregate exposed. The ordinary fiber brush will not answer if the surface is permitted to get too hard. A brush about 4 in. wide, made by clamping together a sufficient number of sheets of wire cloth, has been found to be very effective.

The appearance of a brushed finish can be improved by washing with a diluted solution of acid applied with a brush. The acid thoroughly cleans the surface of the aggregate, thereby intensifying the color and texture of same. The treated surface can be made of any desired color by the proper selection of colored aggregates, but care should be taken to make sure that the materials used are not affected by the acid. The surface should be thoroughly washed after the acid treatment as otherwise it will have a mottled, streaky appearance.

99. Tooled Finish.—Concrete surfaces may be finished by tooling by any of the methods employed for dressing or finishing

natural stone. Only small-sized aggregate should be used in the facing material, as it is hard to dress and to obtain uniform results on surfaces where large angular stones are encountered. The concrete as a rule should be thoroughly hardened before tooling, especially if sharp clean surfaces are desired.

A variety of surface effects may be obtained by tooling, as the effect produced in any given case depends upon the kind of tool used. Some variation in the appearance of the finished surface may also be obtained by the manner in which the tool is handled.

Courtesy of Mr. H. F. Porter, Mfg. Editor of "Factory."

FIG. 381.—Bushhammering concrete exterior. Harvey Hubbell Co.'s building, Bridgeport, Conn.

By striking a perpendicular blow no lines or marks are left in the surface, whereas with a glancing blow, tooth marks are left which can be made parallel to each other or at various angles. Tooling cannot ordinarily be performed satisfactorily on gravel concrete, as the pebbles will be dislodged before being chipped.

Bush-hammering (Fig. 381), either by hand or with a pneumatic tool, is the most popular method used in tooling concrete surfaces. The best results are obtained on surfaces which are thoroughly hard. The concrete should preferably be about two months old, although if it is allowed to stand too long the labor involved will be unnecessarily great.

Tooling is rather expensive and slow work, but an excellent finish can be obtained if the work is properly done.

100. Sand-blast Finish.—A sand-blast finish is very much the same in appearance as that obtained by brushing the concrete surface while it is green. Sand blasting produces a granulated finish somewhat similar to sandstone but not so uniform, because the aggregates are likely to be brought out irregularly.

The concrete should be thoroughly hardened before sand blasting, especially if sharp edges and surfaces of a fine, uniform texture are desired. All pronounced ridges should be removed by tooling previous to sand blasting since, in attempting to remove these ridges with the sand blast, the surface on either side is apt to be cut too deep and depressions in the surface intensified or made more prominent instead of being erased. All strips nailed to the forms to make moldings, joints, and courses should be left in place while the surface is being sand blasted, as otherwise the sharp angles and edges will be rounded off.

The following is taken from a pamphlet on "Concrete Surfaces" issued by the Universal Portland Cement Co.:

"Upon a smooth, dense, thoroughly hardened concrete surface a $\frac{3}{8}$-in. nozzle may be used, but under ordinary conditions $\frac{1}{4}$-in. or even $\frac{1}{8}$-in. nozzles have been found to give the best results. A clean, sharp, thoroughly dried silica sand or crushed quartz is most effective for sand blasting; and for use with a $\frac{1}{4}$-in. nozzle the sand should be screened through a No. 8 screen, and through a No. 12 when a $\frac{1}{8}$-in. nozzle is used. The best results are apparently obtained on a thoroughly hardened concrete surface at least a month old, and for such work a nozzle pressure of from 50 to 80 lb. will be required."

101. Colored Concrete Surfaces.—The most satisfactory concrete surface of a given color and texture can be obtained by properly finishing a surface faced with a mixture composed of cement and an aggregate of the proper size and color. The color is obtained from the exposed aggregate, and not by adding coloring matter to the mixture. The successful production of the desired surface is also dependent upon the proper grading, proportioning, mixing, and placing of the materials and upon the method of finishing the surface.

Where special or expensive materials must be employed to obtain the desired finish, they may be used only in the mixture applied as a facing to the exposed surface. Such a facing is usually 1 to $1\frac{1}{2}$ in. thick. The best method of placing the facing material on vertical surfaces is by the use of a metal facing form or mold, con-

sisting of short lengths of iron plates 8 to 10 in. wide and about 6 ft. long with three angle irons riveted to each plate. The upper end of the long plate should be provided with handles and slightly flared to assist in depositing the material. The metal form is placed against the wall form with the handles up and the outstanding legs of the angles tight against the wooden form. The space between it and the back of the wall is filled with ordinary concrete backing and the 1-in. or $1\frac{1}{2}$-in. space between the metal form and the face form is filled with facing material. The metal form should not be permitted to remain until initial set takes place, but should be frequently raised to prevent the formation of seams between the facing and the concrete backing.

Where aggregates of the desired color are not available, satisfactory results may be obtained by what might be termed body coloring and surface coloring. Only mineral colors should be used since the ingredients of cement have a strong and generally injurious chemical action upon most of the ordinary pigments. There is also danger in body coloring of impairing the strength of the mixture if very much coloring matter of any kind is used. Body coloring need apply only to a facing mixture.

There are a number of specially prepared paints and cement stains on the market that apparently give splendid satisfaction as a surface coloring. The results to be obtained with ordinary oil paints are questionable and should not be used upon concrete surfaces.

CHAPTER XXIII

WATERPROOFING OF CONCRETE

102. Methods Employed.—The various methods used to render concrete waterproof may be roughly classified as follows:

(1) Accurate grading and proportioning of the concrete materials in order to secure a concrete so dense as to be waterproof.

(2) Applying waterproof coatings or washes to the concrete after it is in place.

(3) Mixing foreign substances (*integral* waterproofing compounds) with the concrete.

(4) Surrounding the concrete structure with layers of waterproofing materials, such as asplalt and felt.

It is often advisable to combine two or more of these methods.

103. Dense or Impermeable Concrete.—The method of proportioning and grading the concrete materials in order to obtain an impermeable concrete has been discussed in Art. 34 in connection with the treatment of roof surfaces.

The subject of waterproofing concrete by this method is well covered in the recent report of the Bureau of Standards, Department of Commerce and Labor, Washington, D. C., known as Technologic Paper No. 3. The tests reported were made to determine the absorptive and permeable properties of Portland cement mortars and concretes, and, at the same time, to determine the necessity for and the value of so-called *damp-proofing* and *waterproofing* mediums. All the waterproofing compounds tested and reported were such as were to be incorporated in a mortar or concrete and are known as *integral* compounds. No tests were made on felt papers or similar materials coated with bitumen. The summary to the above-mentioned report follows andindicates that if the work of proportioning and grading the concrete materials is properly carried out, concrete can be made water-tight under ordinary conditions without the addition of waterproofing compounds.

"Portland cement mortar and concrete can be made practically water-tight or impermeable (as defined below) to any hydrostatic head

up to 40 ft. without the use of any of the so-called 'integral' waterproofing materials; but in order to obtain such impermeable mortar or concrete considerable care should be exercised in selecting good materials as aggregate and proportioning them in such a manner as to obtain a dense mixture. The consistency of the mixture should be wet enough so that it can be puddled, the particles flowing into position without tamping. The mixture should be well spaded against the forms when placed, so as to avoid the formation of pockets on the surface.

"The addition of so-called 'integral' waterproofing compounds will not compensate for lean mixtures, nor for poor materials, nor for poor workmanship in the fabrication of the concrete. Since in practice the inert integral compounds (acting simply as void-filling material) are added in such small quantities, they have very little or no effect on the permeability of the concrete. If the same care be taken in making the concrete impermeable without the addition of waterproofing materials as is ordinarily taken when waterproofing materials are added, an impermeable concrete can be obtained.

"The terms 'permeability,' 'absorption,' and 'damp-proof' should not be confused. A mortar or concrete is impermeable (not necessarily damp-proof), as defined and used throughout this report, when it does not permit the passage or flow of water through its pores or voids. The absorption of a mortar or concrete is the property of drawing in or engrossing water into its pores or voids by capillary action or otherwise. If the pores or voids between the grains or particles or in the individual grains are sufficiently large and connected from surface to surface of the wall, the concrete will be permeable to water. If the pores or voids are very minute, but connect one with another, theoretically they may act as capillary tubes, absorbing or drawing in and filling themselves with water; but the capillary forces will tend to hold the water in the pores and will prevent the passage or flow of water, even though one surface of the wall may be exposed to a considerable hydrostatic pressure. For all practical purposes a wall under such conditions would be considered perfectly water-tight and impermeable, although it may be highly absorptive. If these minute pores do act as capillary tubes and are never minute enough to prevent capillary action, the moisture either as water or water vapor would in time penetrate entirely through and fill a concrete wall, no matter what the thickness or composition. In such a case the capillary forces would not permit an actual flow of water, but these forces may carry moisture, entirely filling the wall, and, unless evaporation is retarded, the opposite face of the wall would appear dry. In such a case the concrete would be considered impermeable, but not damp-proof.

"The damp-proofing tests as conducted would indicate that Portland cement mortars can be made not only impermeable but damp-proof as

well, as defined above, without the use of any damp-proofing or waterproofing compound. However, these tests should be interpreted with caution, as the evaporation may have been sufficient to care for the slight amount of moisture coming through the test pieces without indicating on the filter paper. Thus it cannot be stated that if a material were used which was damp-proof according to this test, if used as a basement wall, one surface being constantly exposed to moisture and the other surface in an inclosed room where there would be little or no circulation of air, that the interior surface would not appear damp and the atmosphere become saturated with moisture. The tests of coating materials as damp-proofing mediums can be considered as only preliminary, but the results, considered along with the chemical discussion, throw some light on their comparative merits. The mortar used in these test was, perhaps, too coarse and too absorptive for a fair test. The purpose of the rough surface was to test the flowing qualities of the coating, and it would seem that many of the failures may be due to the poor or imperfect spreading and adhesive quality. Several of the compounds deteriorated and proved their unfitness for the purpose intended.

"Well-graded sands containing considerable graded fine material are preferable for making impermeable concrete, but if such is not to be had, fine material in the form of hydrated lime, finely ground clay, or an additional quantity of cement will be found of value.

"Where Portland cement mortar is used as a plaster coat, if sufficient cement be used and the sand contains sufficient fine material (or a fine material be added) and the mortar be placed without joints and well troweled (care being taken not to overtrowel, which may cause crazing), the coating will be effective as an impermeable medium without the use of any waterproofing compound.

"As a precaution, under certain conditions, it is undoubtedly desirable to use bituminous or similar coatings, even on new work, as protection where cracks may occur due to settling of foundation or expansion and contraction caused by temperature changes. In large or exposed work it is practically impossible to prevent some cracks, but where cracks can be prevented no coating whatever is required to make the structure impermeable.

"The permeability of Portland cement mortars and concretes rapidly decreases with age.

"None of the integral compounds tested materially reduced the absorption of the mortars before they were dried by heating at 212° F. Thus they would have little or no practical value. But some of the so-called integral waterproofing compounds did decrease the absorption obtained after drying the mortars at 212° F., and the rate of absorption was much slower in these cases. The addition of hydrated lime and clays seemed to have little or no effect on the absorption.

"The addition of any of the compounds tested to a mortar in the quantities as used in these tests does not seriously affect the compressive or tensile strength. The addition of the inert void fillers to mortars, as used in these tests, up to 20 per cent of the volume of cement increases the compressive strength."

Efflorescence is the term applied to the whitish or the yellowish accumulations or stains which often appear on concrete surfaces exposed to the action of the weather, and which are due to the leeching out of lime or other chemical salts. The prevention of efflorescence seems to lie in the direction of more impervious concrete. Great care should be exercised at joints and in depositing materials so as to preclude superficial cracks, even of minute size. The more nearly waterproof the concrete, the less efflorescence will show on the surface.

104. Waterproof Coatings or Washes.—Various methods of treating the surface of concrete have been employed to increase the water-tightness. The main objection to the use of any of these methods lies in the fact that a coating is usually rendered worthless by any minute crack in the concrete.

The materials employed as surface coatings may be grouped in the following classes:

(1) Alum and soap mixtures applied in alternate coats, popularly known as "Sylvester" process.

(2) Alum, lye, and cement washes.

(3) Cement grout, with or without the addition of water-repellents.

(4) Paraffine and other mineral bases, applied cold in solution or prepared in melted condition.

(5) Miscellaneous materials of unknown composition and sold under trade names.

(6) Specially prepared bituminous products.

All of the above, except the materials of class 6, are applied to the surfaces directly exposed to the action of water. In the case of class 6, however, to application is made to the inner surface of exposed building walls, the function of bituminous products in this position being not only to damp-proof but to serve as an insulating film against rapid changes in temperature and, in some cases, to replace furring and lathing since plaster may be directly applied to such coatings.

Before applying any coating it is important that the surface be clean, free from foreign matter, and in most cases dry. The

surface should also be thoroughly chipped (or wire-brushed) not more than two days prior to the application of the waterproof coating. The coating should be homogeneous, continuous, sound and uniform; and careful connection should be made to the preceding day's work.

The well-known Sylvester method is probably the most effective method of rendering set concrete waterproof by means of coatings or washes. The alum and soap penetrate the pores of the concrete forming insoluble compounds due to chemical action between the alum and soap solutions, and these compounds prevent percolation. The proportions generally used are three-quarters ($\frac{3}{4}$) of a pound of soap to one (1) gallon of water, and two (2) ounces of alum to one (1) gallon of water, both substances to be perfectly dissolved in water before using. The temperature of the air should not be less than 50° F. at the time of application of the washes. Both solutions should be applied with a soft, flat brush (one for each solution), the soap solution boiling hot, and the alum solution at 60° to 70° F. The soap solution is applied first and allowed to dry—usually for 24 hours—then the alum solution is applied and allowed to dry for the same length of time. This constitutes one treatment, and as many treatments may be given as is necessary. The solutions should be well rubbed in, but care must be taken to avoid frothing in applying the soap wash.

105. Introduction of Foreign Ingredients.—There are two methods of rendering concrete water-tight by the introduction of foreign ingredients: (1) adding a single inert void-filling substance; and (2) adding one or more substances which by action upon the cement or between themselves may produce a void-filling material. Foreign ingredients may also be designated by their capillary attraction for water—such as lime, puzzolan cement, and clay—or by their capillary repulsion for water, such as alum and soap, wax, and a number of proprietary compounds.

Bulletin No. 46—Office of Public Roads, Washington, D.C.—explains a very simple method of damp-proofing concrete by the incorporation of mineral oil residuum with the ordinary concrete mixtures. It also describes the application of oil-mixed Portland cement concrete to several much-used types of structures in which a damp-proofed building material would be of benefit.

106. Layers of Waterproofing Material.—The waterproofing of concrete by the use of layers of waterproofing material is known

as the *membrane* method, and, strictly speaking, is not a method of rendering concrete impervious. Layers of waterproofing materials used in this method range from ordinary tar paper laid with coal-tar pitch to asbestos or asphalted felt laid in asphalt. Both the cementing materials and the fabrics should be elastic and durable, and should retain these properties throughout the range of temperature to which they may possibly be subjected after being placed in the work. In this connection the student should read Art. 35 which treats of separate roof coverings.

It is difficult to make a bituminous sheet adhere to a surface that is either too rough or too wet, or to one possessing too fine a glaze due to richness of cement surface. On this account all surfaces to be waterproofed should be as smooth as possible and should be clean and dry. The concrete should be allowed to set thoroughly, and all uneven surfaces should be leveled up with a coat of cement mortar. Sharp projections on the masonry should be removed or they will puncture the waterproofing. Continuity of work is extremely important.

CHAPTER XXIV

CONSTRUCTION PLANT

By A. W. Ransome,
Vice-President of the Ransome Concrete Machinery Co.

107. General Discussion.—Positive laws cannot be laid down under which to select plant for any and all types of construction, yet it is possible to formulate a few general rules to guide in plant selection, to give examples of typical plants adapted to broad classes of work, and to indicate a few of the possible variations from the plants selected as typical. This is the aim of the chapter. A study of the plants illustrated will give a fairly comprehensive idea of what has already been done by practical and successful workers in the field, and should result in helpful suggestions.

Inasmuch as plant is in reality but a substitute for labor, it would seem obvious that no more should be invested in plant than will yield a good return. This relation between plant and labor is apparently ignored in many instances, and plant charges are incurred out of all proportion to the volume of work to be done. Such a comparison, however, whether made directly or indirectly, between hand labor and the proposed plant, or between this and that plant, must ultimately be made if the selection is to stand the test of experience.

The selection of plant and the purchase of machinery has to a large extent been more or less haphazard. Contractors and engineers, experienced and successful men, have been slow to awake to the possibilities for loss or gain afforded by plant selection; but it is nevertheless deserving of careful study.

There seems to be a strong tendency toward excess in plant expenditure, and a fact worthy of note is the tendency toward simplicity in plant upon the part of those engineers and contractors whose experience and success in the field entitles them to be considered as leaders.

It has been stated again and again in such books and technical papers as treat upon the subject, that manufacturers of plant

tend to overrate their machines, and that the careful engineer should purchase plant with possibly double the rating actually required. This practice is wasteful, and if carried out in all directions will spell failure. It is a simple matter to estimate the operating speed upon which the manufacturer bases his ratings, and from this to determine whether or not you can maintain such speed under the conditions which will prevail on your work.

The advisability of machine mixing wherever practicable will be conceded by anyone devoting thought to the subject. There are, for instance, well-defined advantages of economy in labor, in greater uniformity of product, and in greater speed in construction. An efficient concrete plant is also an excellent pacemaker and this fact alone is an important factor in making for general adoption of mechanical mixing.

There are, of course, many small jobs on which the possible savings would be more than offset by the cost of installation of a concrete mixer. For our purpose, however, we may very well leave such jobs out of consideration, and consider only work of a size sufficient to warrant plant of one kind or another.

108. The Mixing Cycle.—In considering the possibilities of a plant, it is best to consider the cycle of mixing as complete in three operations—namely: charging, mixing, and discharging—and for our present purpose we shall consider *plant* as referring to a mixer with a capacity of ½ cu. yd.

In charging a mixer a limit is set by the physical laws governing the flow of materials from one container to another, and true effectiveness is more or less attained according as this limit is more or less nearly approached. In illustration of this principle consider a half-yard capacity machine equipped with feed chute (Fig. 382). To handle the full batch, proportions 1:3:6 will require, with the ordinary barrow (Fig. 383) containing 3 cu. ft., a procession of 7 men as indicated (Fig. 384); and for each man in the procession to wheel up, discharge his load into the machine, and make way for the next, will occupy on the average 6 seconds, making the total time for assembly of the batch 42 seconds. This time may be substantially cut, under favorable conditions, by the use of carts containing 6 cu. ft., instead of the smaller barrows; or by the substitution of a charging car. Similarly, where a fixed charging hopper (Fig. 385) is used, the controlling factor (since the dumping time for barrows can be overlapped on the mixing and discharge operations) is the time required for the flow

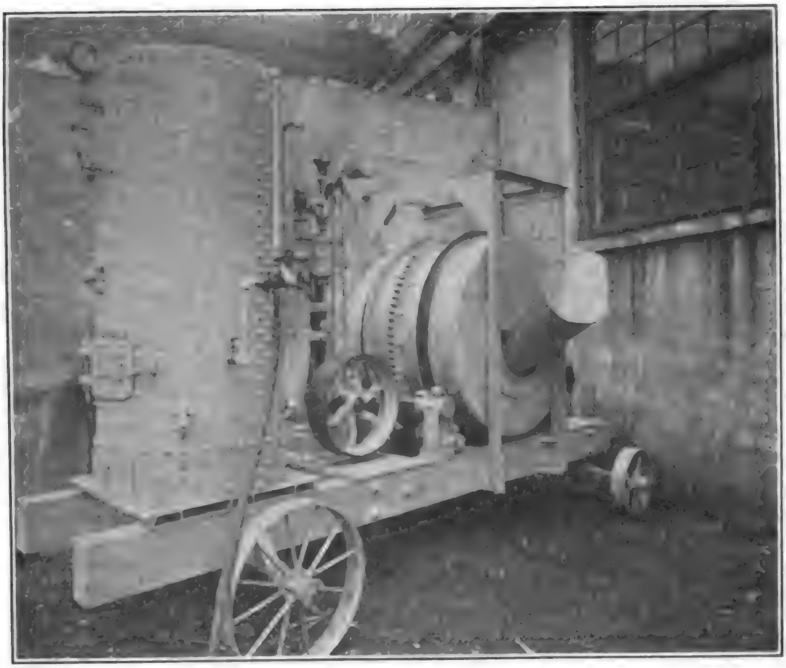


FIG. 382.

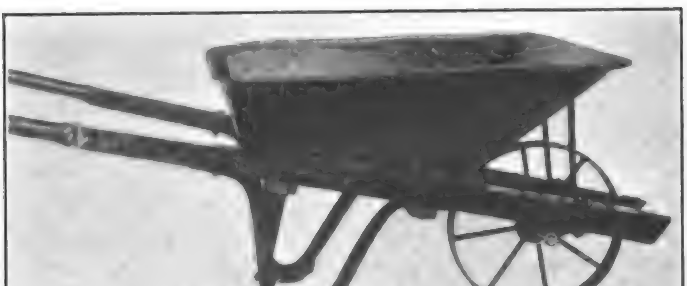


FIG. 383.

of material from hopper to mixer, averaging 10 seconds. With a pivot charging hopper (Fig. 386), there must be added the time required for elevating the hopper, say 10 seconds. This time may be saved by installing slightly heavier power equipment, to permit overlapping this operation on the mixing and discharging operations.

The foregoing discussion indicates sufficiently for our purpose the limiting factors affecting the charging of the mixer, the mixing and the discharge—limits set being taken from personal observation on actual operations. The purpose of this short analysis will be manifest when we come to a consideration of co-ordination of plant.

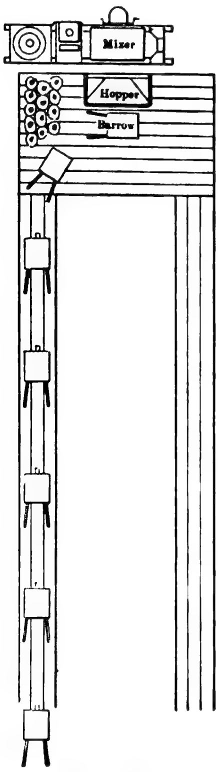


Fig. 384.

Thorough mixing of materials is quickly accomplished in any standard mixer, 30 seconds being ordinarily enough for the purpose. Modern mixers are driven 16 to 20 revolutions per minute, and at each revolution effect 4 or more complete turns of materials, from which it will be appreciated that 30 seconds allowed for the mixing operations is sufficient. It may be thought that good results are not obtained in so short a period, yet the fact remains that this speed has been maintained day in and day out, the resultant product standing favorable comparison with similarly-proportioned concrete mixed under other conditions for longer periods. There is, unfortunately, a tendency upon the part of engineers and contractors to consider their work done when the mixer is purchased and installed. The purchase and installation of the mixer is only the beginning, and the organization of the mixing crew, the regulation of the proportions of water, and the order of feeding materials in the drum should be intelligently directed, rather than left to rule of thumb as applied by the ordinary operator.

For quick and thorough mixing, water should first be fed into the mixer, and this water should be uniform in amount. The speed of the mixer drum can be varied with advantageous

results according to the character of material and the consistency of the mix. In other words, it pays to study your plant in order to learn its limits. The possibilities are a batch every minute, and one's aim should be the closest possible approximation to the possible, consistent with available labor and with the available amount for plantage.

To discharge a mixer requires 10 to 20 seconds, depending upon the consistency of the batch and the size thereof. An important factor in discharge is the speed of the mixing drum, as a varia-

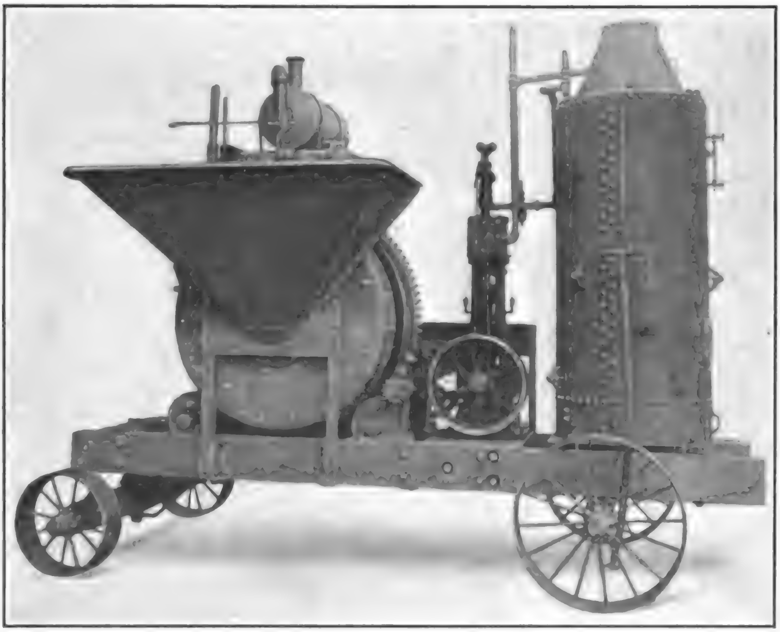


Fig. 385.

tion of 5 to 8 revolutions of the drum may result in a variation in discharge of 10 to 30 seconds.

We have touched at some length on the above details of mixer operation, as they are really important guides in plant selection. This knowledge of plant operation should extend to and cover thoroughly the effect of varying drum speeds on mixing and discharge operations and the effect of varying amounts of water on these operations. As an indication of results possible in this direction, let us cite the following:—with batches of uniform consis-

tency, drum speeds of 12, 14, 16, 18, 20, 21, 22, 24 revolutions resulted in the discharge of the complete batches as follows, in seconds: 26, 22, 18, 15, 12, 10, 12, 16, indicating clearly maximum results at 21 seconds. Similarly with constant drum speed and varying quantities of water the following results were noted:

Gals. of water per 20 cu. ft. batch......	10	15	20	25	27	29	30	31	33	38
Time of discharge in seconds...........	18	22	20	15	12	11	10	11	15	22

A similar series of observations were made as regards the effect of varying amounts of water on time of mixing and on power re-

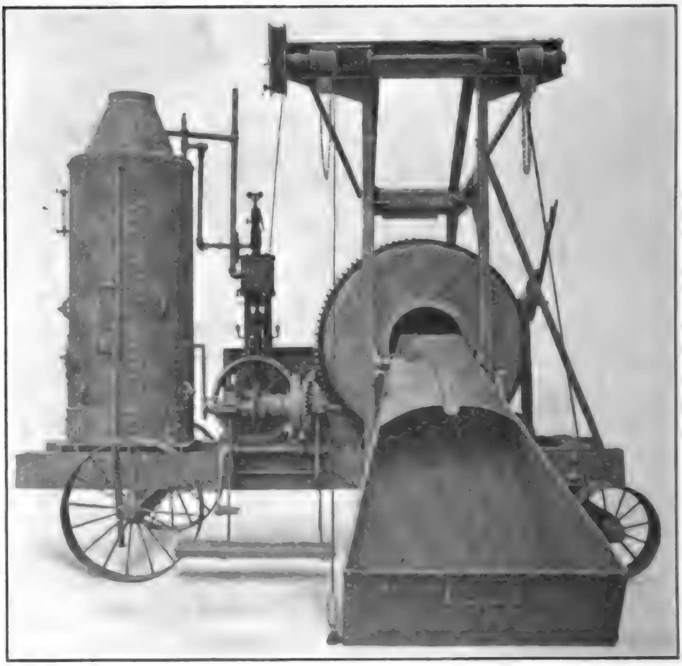


Fig. 386.

quired to drive the mixer, showing acceleration of mixing process as more water was added up to 1½ gal. to the cubic foot. The power consumed decreased with the addition of water to the amount of ½ gal. to the cubic foot, remaining stationary up to 1 gal. to the cubic foot, and then gradually increasing as more water was added.

109. Plant Economics.—Aside from *first cost of plant*, various

other elements must be carefully considered—namely: *cost of installation; cost of operation; cost of maintenance; cost of removal;* and *interest on the investment.* All these items must be considered as against the resultant saving in labor, and the salvage value that the plant will have when the work is completed.

In general the plant best suited to the work is cheapest, regardless of whether or not first cost is a few dollars more than something less suited to conditions. *First cost* has perhaps a less important influence on final results than *cost of operation and maintenance*, and in many cases the higher salvage return will offset a higher first cost to a large degree.

Cost of installation includes freight, cartage, and erection—all of these elements varying with the character of the plant, location of the work with respect to the source of supply, etc.

Cost of operation depends primarily upon plant arrangement and organization—principally the latter. Properly handled, concrete plant becomes an important factor in setting the pace for the work. The plant should preferably be of a size and capacity to permit of continuous operation during working hours, and the balance of the organization and equipment be so coordinated as to permit this result.

110. Conditions Governing Plant Design.—The character and arrangement of the plant depends, to a large extent, upon local conditions, such as the contour of the ground and the character of construction; also the manner in which the materials are to be delivered to the site, whether in cars, or wagons, regularly or irregularly, has an important bearing on the type of plant. Similarly, the matter of total yardage to be placed, of time limit set for the work, of bonus of penalty, all will have a bearing on plant selection.

Other considerations, which may materially affect the selection, are the amount of ground available for material storage, and the time of year during which the operation must be carried on—winter work requiring very different plant arrangement from summer work.

Contour of the ground is principally effective in determining the location of the plant with respect to the work and the storage of materials. For example a steep slope will often make advisable a system of overhead bins with gravity feed, which under other conditions would not be advisable.

The general layout of the work will usually be the determining

factor in the adoption of means for handling mixed concrete, subject of course to the modifications imposed by total yardage, etc. It may make for the adoption of two or more separate installations, rather than one central plant, or it may cause the adoption of a portable plant, rather than a stationary one.

Delivery of materials is principally effective in determining the arrangements for storage of raw materials.

Total yardage, time limit, etc., are generally the controlling factors in determining the amount available for plantage.

Having determined the amount available for *plantage*, the problem changes more or less and becomes one of proper co-ordination of the various elements, a fitting of one into the other, a balancing as it were. Careful study of plant is well repaid in results, and a well-balanced plant is invariably profitable. The installation of mixer capacity for 40 cu. yd. per hour, in connection with charging facilities for only 20 yd. per hour, or with provision for handling mixed materials at the rate of 20 yd. per hour, is waste. The illustration might be elaborated, as many such conditions occur quite often in actual practice.

111. Types of Mixers.—Considered broadly, mixers may be divided into *drum mixers; trough mixers; gravity mixers;* and *pneumatic mixers.*

Drum mixers may again be divided into *tilting mixers* (Smith Type) and *non-tilting* (Ransome Type). In the former class the mixing drum is mounted on a swinging frame, and the discharge of mixed materials is accomplished by a tipping of the frame and drum. In the latter class mixed materials are drawn out through a chute inserted into the drum.

Trough mixers, as a whole, may be designated as *paddle mixers,* though the paddles may vary in form from a broken worm through the various stages, to the continuous worm, and the conveyor flight may be single or double, of varying or uniform pitch.

Gravity mixers are of the same general characteristics, depending for success upon a series of deflections, chains, pegs, or conical hoppers, for the mixing action. They are not adapted to building work in any case, and do not deserve serious consideration here.

Pneumatic mixers include the various types of penumatic mixers developed during the past two or three years by Wm. L. Canniff, A. W. Ransome, McMichael, and Eichelberger. In the Ransome and Canniff mixers the materials are first mixed by air in a container, and the mixed concrete then forced out

through pipes to its ultimate destination. In the McMichael and Eichelberger machines, the materials are assembled in a container and forced through pipes without permixing. These latter machines depend for successful results upon such mixing action as may take place in transit through the pipe.

The above grouping of mixers, being largely based upon mechanical details fully described in advertising literature and current technical papers, is not one upon which we need enlarge here. We are interested in the general application of plant to construction work, rather than in any special mechanical device. It will be readily appreciated that for building construction (which is the special object of consideration here) a machine of the drum type is preferable, and our remarks will be mainly devoted to this type of machine, with a mental reservation to the effect that, under special conditions, mixers of other types might be advantageously substituted; and that while we use illustrations of a specific machine, we do so for the sake of clearness.

112. Storage of Materials.—One of the first problems for consideration is the storage of materials. In many cases no special provision need be made; but, on other work, the handling of materials economically warrants more or less investment to this end.

Storage must be provided for enough material to ensure against shut-downs, or operating at part capacity. If space is not available on the site, it should be secured elsewhere; a vacant lot close by will serve for a reserve against emergency.

Whenever possible, the force of gravity should be called into play, and the natural advantage of the site utilized to the fullest extent. Since under ordinary conditions there is double the quantity of stone, as compared with sand, stone should be placed in the most favorable position as regards subsequent handling. Likewise, the position of the sand should have more consideration than that of the cement.

Too little attention is directed to the care of materials upon delivery to the work. Cement is often carelessly protected against the weather, and stone is dumped indiscriminately on the site, resulting in subsequent unnecessary handling or loss due to the admixture of dirt and rubbish. There are very few, if any, jobs where it will not prove economy to carefully level the ground selected for storage, and to cover the ground under the stone pile with 1 or 2 in. of sand, the cost of which will be

saved in increased efficiency of the men who must subsequently shovel the stone.

An excellent plan for material storage is shown in Fig. 387. This plant, consisting of a hopper, elevator, 5-h.p. motor, timber bulkheads and 6 bin gates, involved an investment of apprxoi-

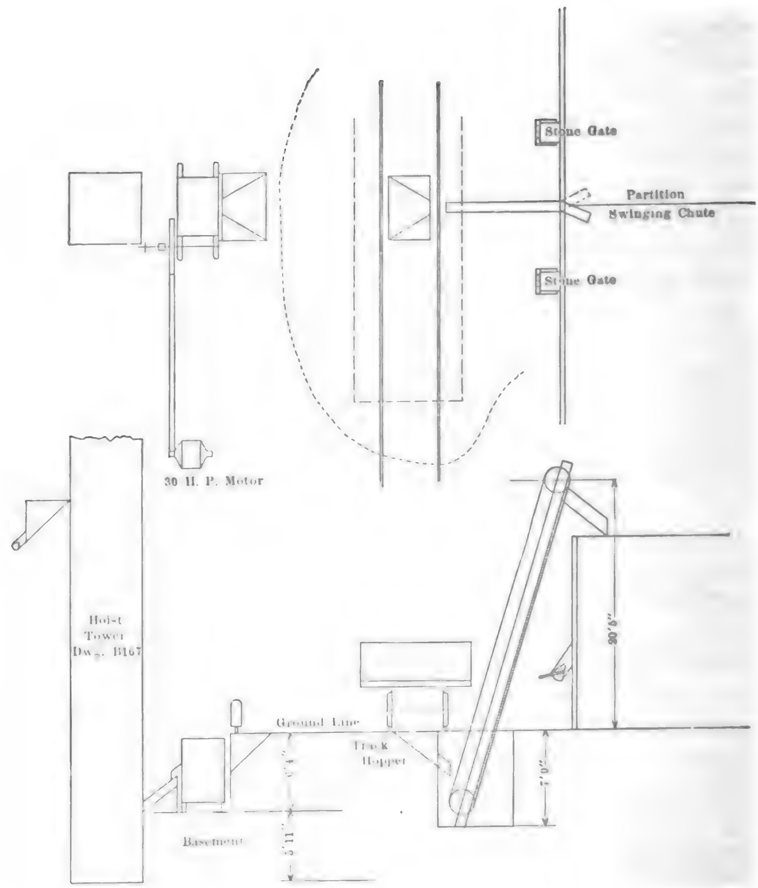


Fig. 387.

mately $500; but all shoveling of sand and stone was eliminated, with the result that the crew necessary for charging the mixer (½ yd. size) was cut from 12 to 6 men, and the unloading of materials delivered in bottom dump cars was cut from 10 to a little less than 5 cents per cubic yard, including cost of electric current and the time of the necessary attendant.

Fig. 388 illustrates storage arrangements and handling facilities as installed in connection with a large building for the Singer Manufacturing Company at Elizabethsport, New Jersey. The available storage space in this instance was a narrow strip 20 ft. wide between the railroad track and the building, of which the work in question formed an extension. A trench was made, extending the length of this available space, and this was sheathed and converted into a tunnel by a covering of 2-in. plank. In the bottom of the trench was laid a suitable track for the operation of a skip car. On the ledger pieces were mounted two saw-toothed measuring hoppers, fitted with wheels.

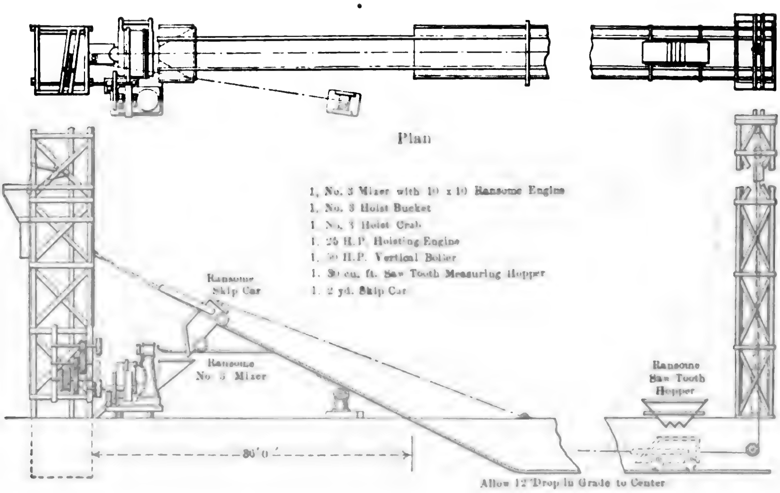


FIG. 388.

Materials were unloaded from the cars and piled over the trench, sand and stone in alternate piles. The saw-toothed measuring hoppers were kept continually against the toe of the sand and stone piles to facilitate charging by breaking down of the face of the piles. Four men in this way handled the materials for about 30 cu. yd. of concrete per hour. Materials were automatically dropped from the measuring hoppers into the skip car as it passed below the hoppers on its way to the mixer. The A frame shown in Fig. 389 has been used to good advantage as a substitute for the trench. With this arrangement one man only was needed to charge the measuring hopper.

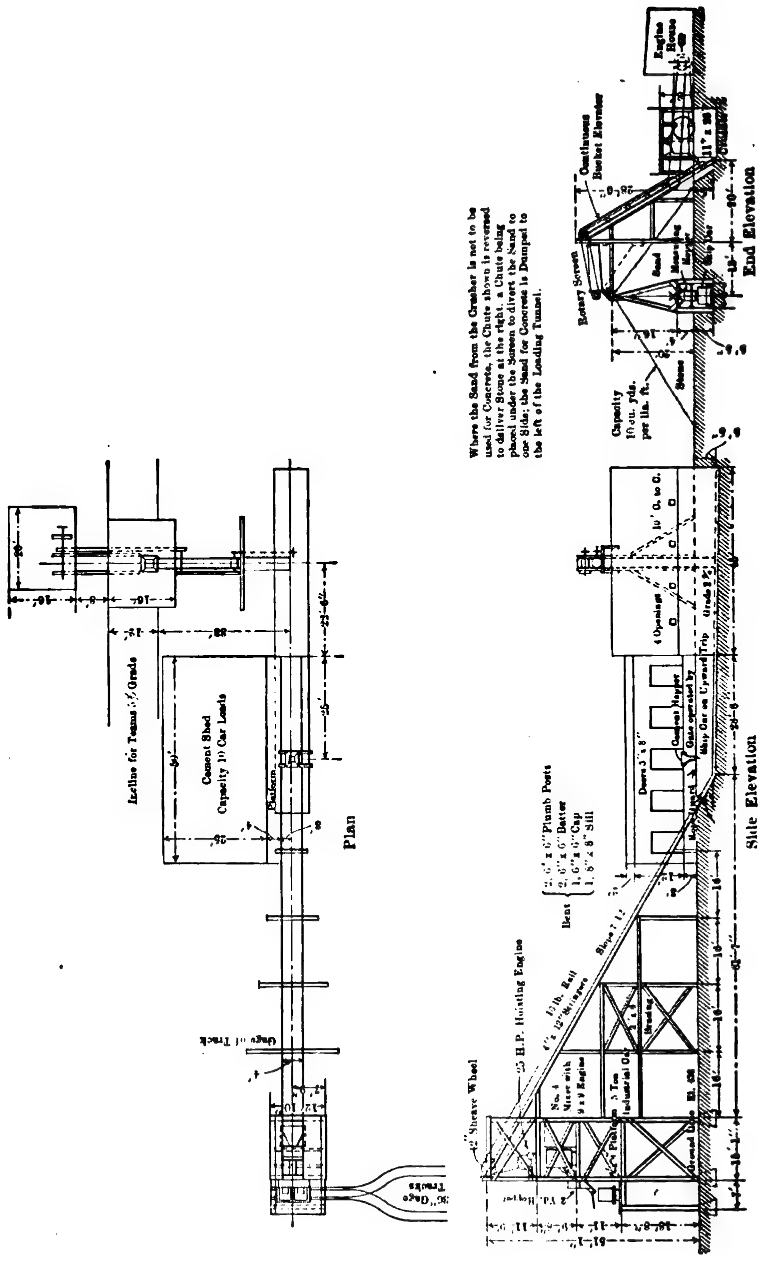

Plan
Side Elevation
End Elevation
Cement Shed Capacity 10 Car Loads
Incline for Teams 5% Grade
Where the Sand from the Crusher is not to be used for Concrete, the Chute shown is reversed to deliver Stone at the right, a Chute being placed under the Screen to divert the Sand to one Side; the Sand for Concrete is Dumped to the left of the Loading Tunnel.
Rotary Screen
Continuous Bucket Elevator
Engine House
25 H.P. Hoisting Engine
12' Sheave Wheel

FIG. 389.

113. Measurement of Materials.—Materials may be measured by volume or by weight, with practice tending strongly toward measurement by volume as being easier, and, with ordinary

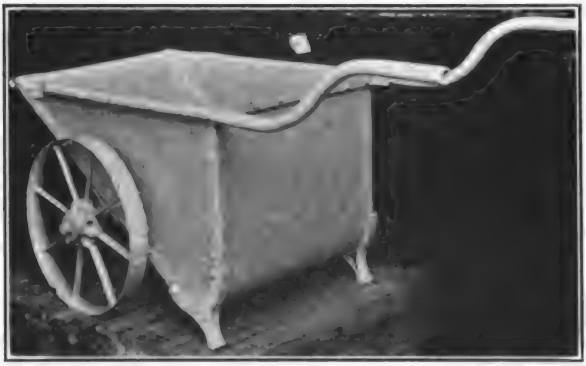


Fig. 390.

precautions, equally accurate. The very common tendency, however, toward the use of the ordinary round or oval tray barrow for measurement is to be avoided, as accuracy is unat-

Fig. 391.

tainable by such means. In all cases, a receptacle used for measuring materials should have a top which is straight and materials should be struck off flush.

An excellent type of measuring barrow is shown in Fig. 390 and a larger 6 cu. ft. capacity cart suitable for this purpose is shown in Fig. 391. The *fixed charging hopper* shown in Fig. 385 is suitable for measurement; but, owing to the large area of the top, slight variations in relative height of materials and top of hopper produce considerable difference in the quantity of materials. An improved type of fixed measuring hopper is shown in Fig. 392. The *pivot charging hopper*, Fig. 386, and *sliding pivot hopper*, Fig. 405, are not suitable for measuring. Where these devices are used, measurement of materials should take place in suitable barrows, carts, or cars. *Skip cars* are not suitable for measuring, and should be used in connection with *saw-toothed hoppers* or other similar devices.

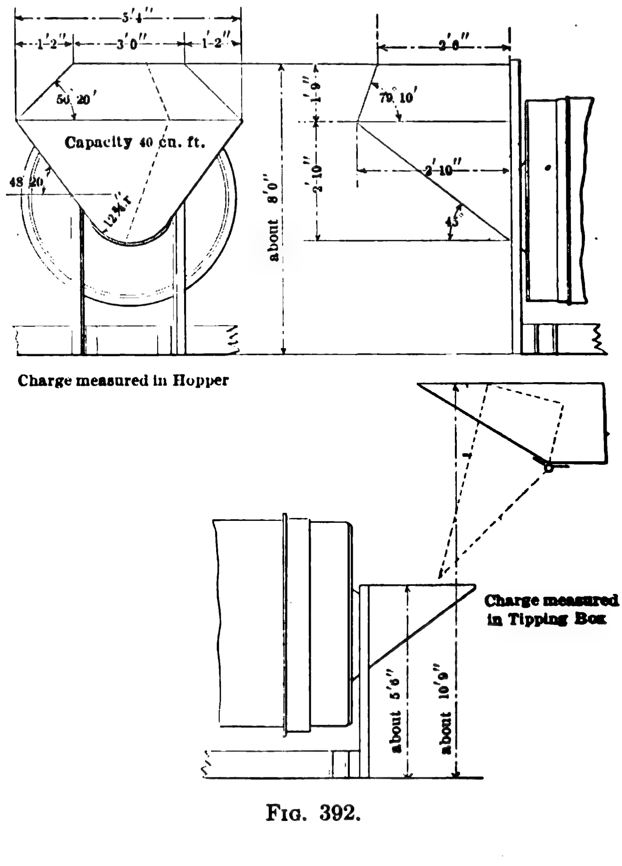


FIG. 392.

114. Placing of Concrete.—Inasmuch as we are considering plants adapted to building construction, a hoist of some sort is an essential part of the equipment. It will ordinarily be found advisable to install the hoist at the beginning of operations, since by so doing the mixer may readily be set so that the operation of charging may be facilitated, principally by cutting out inclines, with resultant saving in labor.

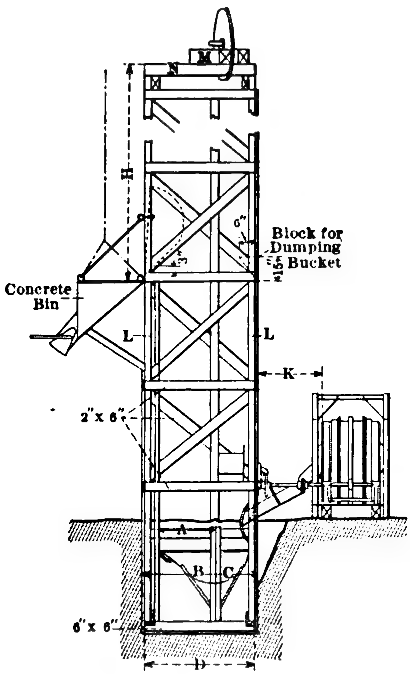


FIG. 393.

A typical hoist is shown in Fig. 393, operating in connection with a mixer, the power being taken from an extension of the mixer shaft. The power equipment of the latter is of sufficient capacity to operate both mixer and hoist at one and the same time. A variation of this plant showing a direct-connected hoist is shown in Fig. 394, but for ordinary conditions the first described arrangement is preferable.

At any desired height a bin or hopper is set, into which the material is discharged by the hoist bucket. From this point

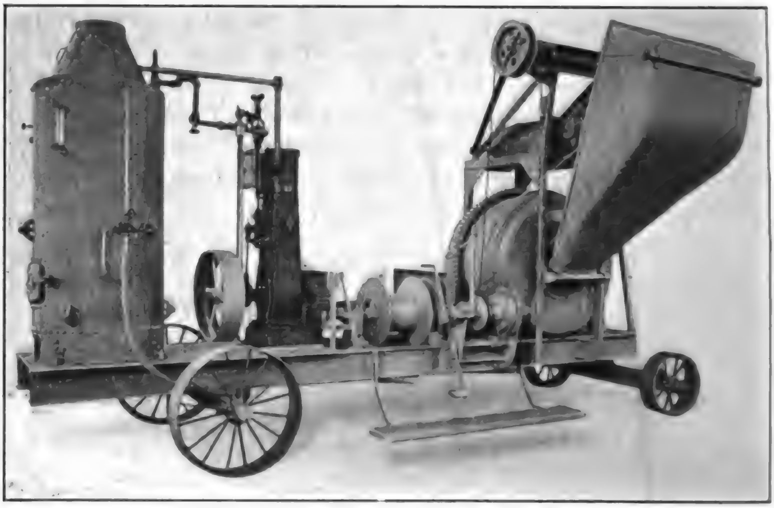


Fig. 394.

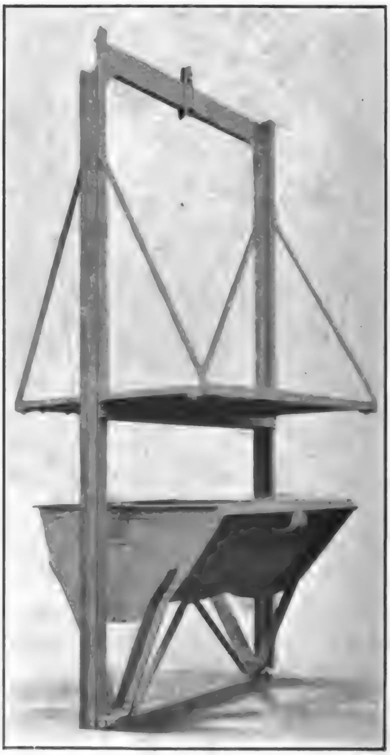


Fig. 395.

distribution may be effected by wheelbarrow, cart, car, or spout, either alone or in combination.

The addition of a platform hoist may be readily secured from a number of manufacturers. A platform hoist is shown in Fig. 395.

In many instances wheelbarrow or cart distribution is demanded by the amount of materials involved, and, where either barrow or cart is used, substantial runways are essential and result in economy. The runways should be of ample width for barrows to pass freely, and a broad turnout should be provided at the concrete bin. Suitable barrows and carts are shown in Figs. 383 and 391.

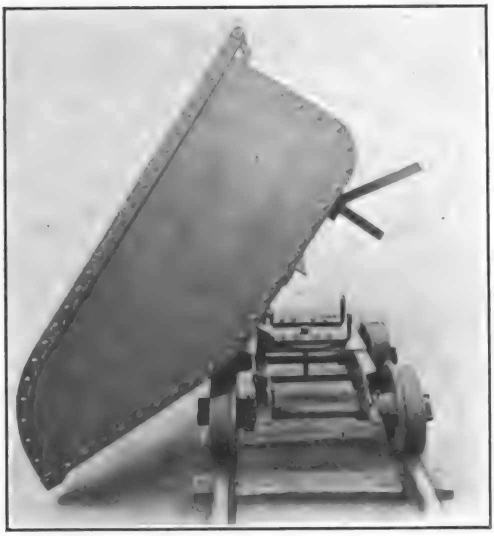


FIG. 396.

Distribution by cars is largely used by the Aberthaw Construction Company, and in Fig. 396 is shown the type of car they have adopted for this work. Fig. 159 illustrates the system of operation.

The handling of concrete through spouts or chutes is of comparatively recent development, and, as in many other developments, there has been a tendency to overdo. Spouting systems have been installed on many buildings where the distribution might have been better done by barrow or cart. The installa-

tion of a spouting system is expensive, and should not be undertaken blindly; nor with expectations of abnormal savings.

Spouting plants may be grouped under *boom plants, guy-line plants,* and *tower plants.* In the former, Fig. 397, the spouting is mounted on a swivelled bracket at the tower end, and the outer end, supported by a boom, moves freely about the work.

Fig. 397.—Boom spouting plant.

A second length of spout ordinarily completes the unit. This type of plant has a greater freedom of movement than either guy-line or tower plants, but is not as free moving as might be desired.

Many ways have been tried to facilitate ready moving of the free end; none of them, however, proving entirely satisfactory.

A suggestion has been made to counterbalance the free end; but this has not as yet been tried out.

In guy-line plants, Fig. 398, the spouting is suspended by ordinary blocks and falls from guy lines or from special cables set up for the purpose. In some cases the outer end of the cable is mounted on a portable tower or A frame, and the blocks and falls are preferably arranged so that necessary adjustments in the line may be made from the ground.

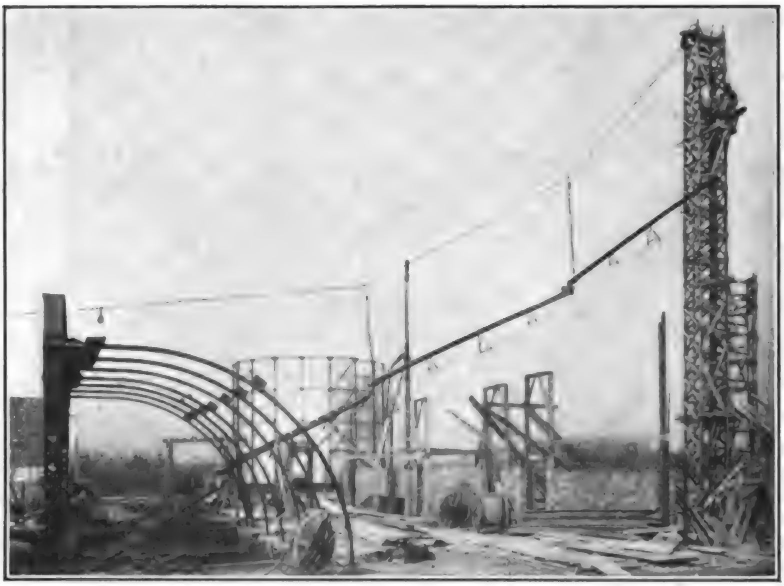


FIG. 398.—Guy-line spouting plant.

In tripod plants, Fig. 399, movable towers are used to support the ends of various sections of spouting.

It has been found by practical experience, that concrete thoroughly mixed, and of proper consistency, will flow on a slope of 18 degrees, with the best results obtained at 23 degrees. These slopes, however, are based upon a rigidly supported chute. Where the spouts are supported from guy lines, the slope must be a little steeper, preferably 27 to 30 degrees. By proper consistency is meant a mixture with approximately $1\frac{1}{4}$ to $1\frac{1}{2}$ gal. of water to the cubic foot of material. There should be just as much water as the material can carry without separation, so

that the stone particles will be carried in suspension in the mass. There should be a sliding of materials down the spout, rather than a rolling.

Various types of spouting have been tried, ranging from round pipe to rectangular troughs. Best results have been secured from the use of 5-in. pipes, or 10-in. open troughs, the latter

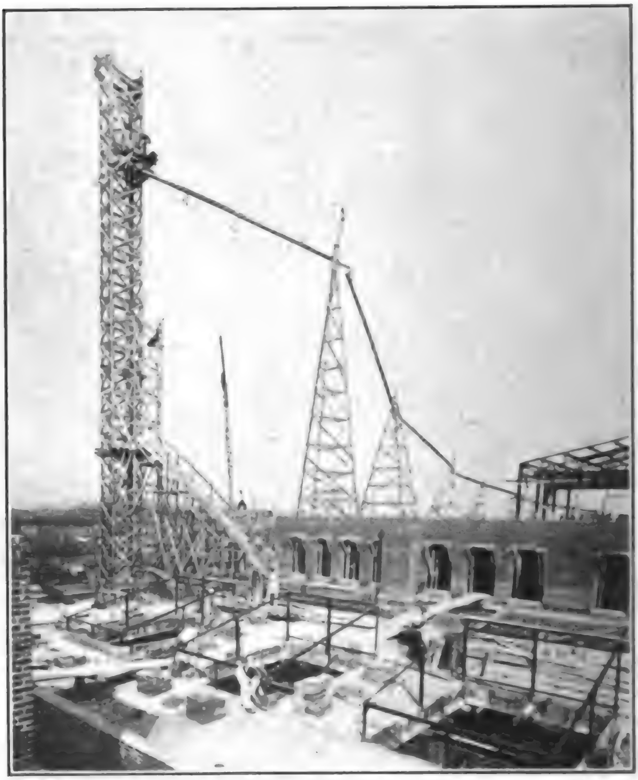


FIG. 399.—Tripod spouting plant.

having the preference for flat slopes, and the former where there is necessity for varying pitch, with a likelihood of steeper pitch than named above.

With open spouting the use of remixing hoppers (Fig. 400), in connection with flexible spouting (Fig. 401), accomplishes satisfactorily the necessary changes in pitch. The greatest items of

expense in spouting plants are first cost, installation, and maintenance.

Maintenance charges are particularly heavy. The ordinary stock spouting which is made of No. 14 gage metal will seldom handle more than 2000 yd. without renewal. This is due to the

Fig. 400.

abrasive action of the material, especially as affecting the rivets which join the various sections.

A recent development is a spout made up of two or more longitudinal sections of the shape indicated in Fig. 402. The various sections are interchangeable, and there are no bolts or rivets extending through the spout, all joint bolts or rivets passing through the flanges, and the various longitudinal joints made secure by fish plates. This type of spouting has the further

FIG. 401.

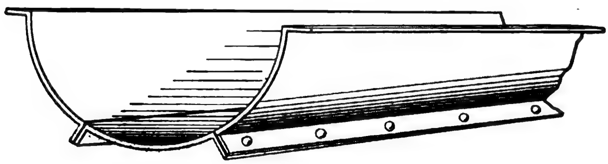


FIG. 402.

FIG. 403.

way of bucket elevator to the bins above the mixer. Once in the bins, it is a gravity process through measuring hopper to mixer. Messrs. Cramp & Company report 289 cu. yd. with this plant in 8½ hours, with a crew of eight men, and covering all handling from wagons and cement storage to delivery bin on hoist tower. The mixer is ½ yd. capacity.

Fig. 405 illustrates an arrangement adopted by the Turner Construction Company on the Bush Stores, South Brooklyn. Materials were delivered to the work in standard cars, and

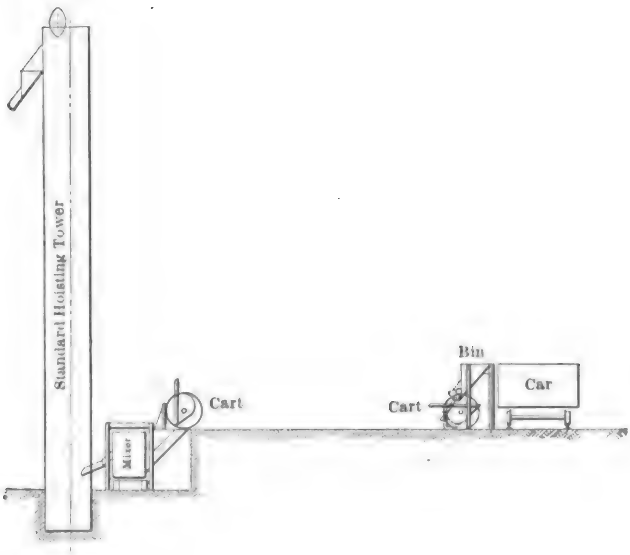


Fig. 405.

unloaded by shovel into special hoppers as indicated. These hoppers were readily portable, and each hopper had a capacity for approximately 1 cu. yd. Materials were drawn off as required into carts with a capacity of 6 cu. ft., and wheeled to the mixer which was set at a lower level so that the fixed hopper was on a level with the ground. The bumping post *A* facilitated discharge of carts into the hopper. Carts were wheeled up against the bumper when a slight lift on the handles did the trick.

Fig. 406 illustrates the plant used in the erection of the buildings for Foster Armstrong Co. at Despatch, New York. The work on these buildings was carried on through the winter

months and the bins indicated provided the readiest means for heating the materials to the desired temperature of 90 to 100° F. An auxiliary measuring tank took care of the salt solution used

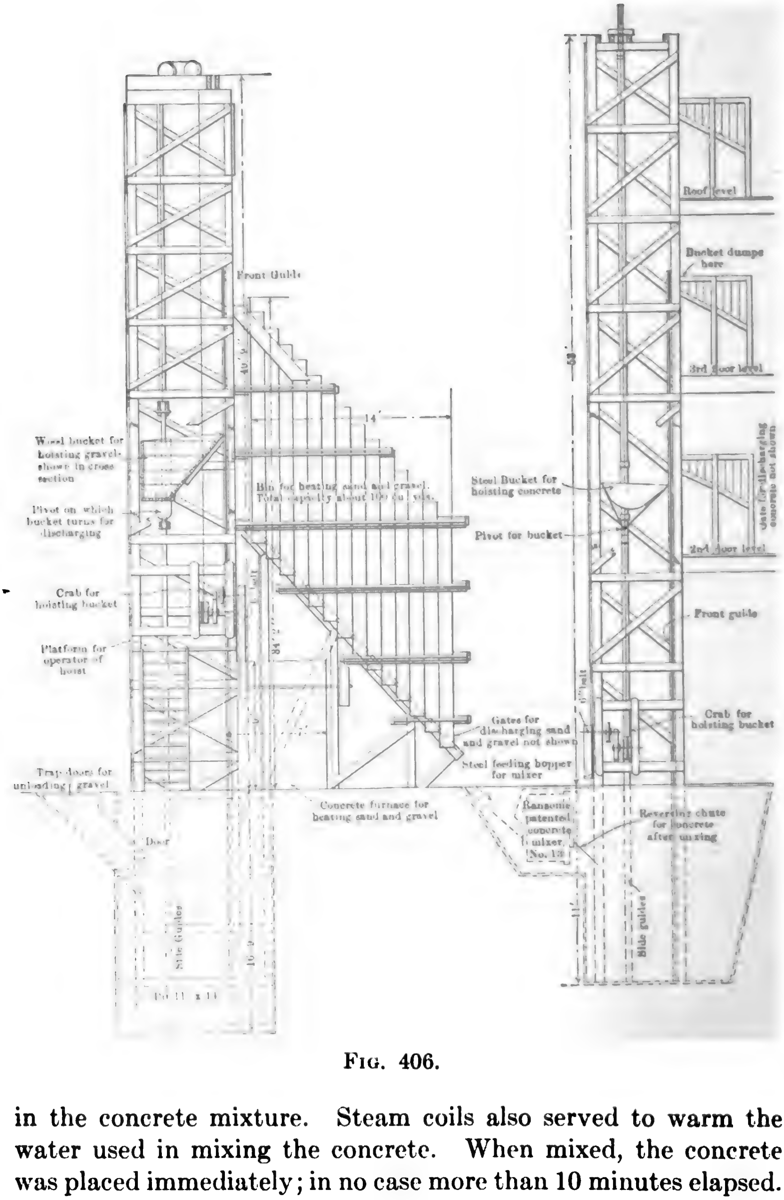


Fig. 406.

in the concrete mixture. Steam coils also served to warm the water used in mixing the concrete. When mixed, the concrete was placed immediately; in no case more than 10 minutes elapsed.

When the concrete had been placed, it was protected against the action of frost by a solid wood covering, blocked up at least 6 in. above the surface of the floor in a manner to permit free circulation of air beneath the covering. Heat was introduced beneath the floor (or in the case of ground floors, beneath the board covering) by means of steam-coils and salamanders, provision being made for the escape of sufficient steam beneath the covering to prevent premature drying out of the concrete. Salamanders were sprinkled freely with water, thus producing the necessary amount of moisture, and small openings were left in the floor slab to permit the warm air to circulate over the upper surface of the floor. The sides of the floor were protected by

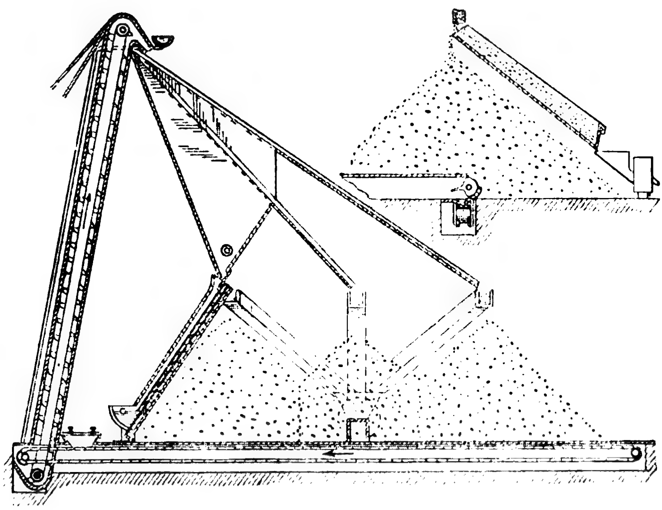


FIG. 407.

canvas curtains which extended downward to the floor next below.

There were placed beneath the floor and beneath the panels on top of the floor, at intervals of 10 ft., self-registering thermometers, which in no case showed lower than 32 degrees. This temperature was maintained until the test cubes which had been allowed to set on the floor and beneath the top covering showed the strength used as a basis for the design.

The extra plant involved in carrying on winter work involves considerable outlay, and work in freezing weather should not be

undertaken without a thorough understanding of all that is involved.

Fig. 407 indicates a more or less elaborate plant, designed for large work, or for cramped quarters. The plant consists of a suitable bucket elevator, designed to handle the entire aggregate. This elevator discharges the materials into trough screens as indicated. While passing through these screens, the coarse materials are washed by a flow of water applied at the head, and the sand is still more thoroughly washed by passing through the water boot and inclined worm. The washed materials are dis-

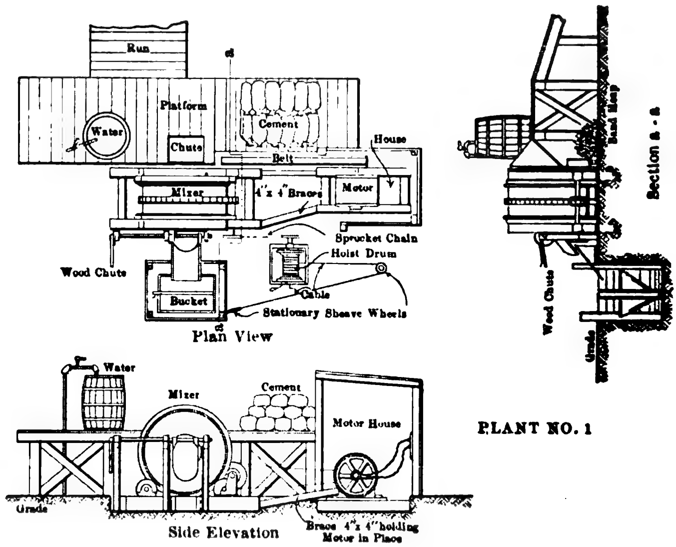


FIG. 408.

charged from the first flight of trough screens upon inclined troughs leading to a fixed measuring hopper at the mixer. The lower end of the troughs is fitted with a gate to control the flow. Such excess material as cannot be cared for by the measuring troughs falls to the ground in piles as indicated. Two belt conveyors operating in tunnels beneath the piles permit ready draft against these reserve piles, delivering materials to the original elevator, and thence to the measuring troughs. With such a plant it is a simple matter to handle upward of 50 cu. yd. per hour with four men. The arrangement shown has been covered by U. S. Letters Patent.

Figs. 408 and 409 illustrate two set-ups of practically the same plant, and illustrate forcibly the importance of proper arrangement. With the arrangement shown in Fig. 409, 17 men handled 72 cu. yd. in 4½ hours. With the arrangement shown in Fig. 408, 15 men handled 87 cu. yd. in 4½ hours, a saving of approximately 13 cents per cubic yard. In Fig. 408 the runway and platform are too small, with the result that the men interfere with each other, and cannot work to advantage. Furthermore, the use of the feed chute involves assembly of the batch in the mixer drum. Water was fed to the machine a bucketful at a time, requiring an extra

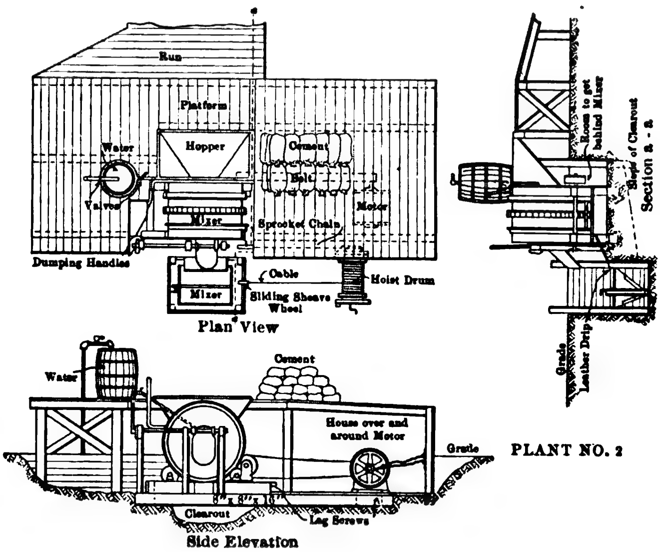


FIG. 409.

man for this purpose. In Fig. 409 a charging hopper is substituted for the feed chute, and both platform and runway are increased in size. Water is fed to the mixer through a pipe, and all operating levers are controlled by one man. Provision is also made to take care of any material working down beneath the mixer, with the result that wear and tear on journals, etc., is reduced.

The difference in cost of the two arrangements amounted to $59.00. The illustrations are taken from actual experience, and indicate results secured under different superintendents.

Problems

34. What items would you charge up against construction work for the use of the plant involved?
35. Enumerate the various operations involved in the erection of the reinforced-concrete skeleton of an office building.
36. State your opinion in regard to the location of stock piles.
37. What considerations should govern the actual laying out of construction plant?
38. What is the sole purpose in the design and use of construction plant?
39. What are the requirements of economic plant design?
40. State the essentials in the proper mixing and placing of concrete under different conditions.

SECTION 3

ESTIMATING

By Leslie H. Allen

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The operation of estimating the cost of reinforced-concrete construction may be divided into two parts: (1) estimating quantities or *taking off*, and (2) estimating unit costs or *pricing*. The estimating of unit costs is by far the more important of these two and this part of the subject will therefore be treated first.

CHAPTER XXV

ESTIMATING UNIT COSTS

Reinforced concrete work may be considered under the following four main divisions:

(1) The concrete itself.
(2) The forms or falsework.
(3) The steel reinforcement.
(4) The finish of the exposed surfaces.

Each of these may be sub-divided into:

(a) Materials.
(b) Labor.
(c) Plant.

Each of these divisions and sub-divisions should be considered separately, and analyzed when making an estimate.

In the prices mentioned in this chapter, the cost of labor is based upon the rates paid in the large cities at the present time, (namely: carpenters 45 cents per hour, laborers 25 cents per hour, carpenter foremen $5 per day) and include the overhead expense of superintendent and timekeepers in charge of the work.

116. Concrete.—With regard to the concrete itself, we shall take into account: (a) the cement, sand, stone, and water; (b) the labor of unloading, mixing, and placing of these materials; and (c) the plant necessary to accomplish this end.

The cost of cement in carload lots is, as a rule, about 85 cents per barrel at the cement mill. If the cement is delivered in paper bags, there is an extra charge of 10 cents per barrel for bags. If delivered in cloth, the extra charge is 40 cents, but this 40 cents is refunded by the mills if the cloth bags are returned in good condition. If furnished in wooden barrels, there is a charge of 40 cents, and the barrel is not returnable. It is usually considered the most economical to buy cement in cloth bags and return the bags when empty. The freight on a barrel of cement on a haul of say 500 miles would be about 40 cents, so that the total cost of a barrel of cement in cloth bags after returning and crediting the bags would be $1.25 per barrel. The cost of testing cement is from 3 to 5 cents per barrel. It is customary among contractors and engineers to have the whole shipment of cement tested at a testing laboratory, and from $5 to $6 per carload is about the usual charge. The cost of unloading cement and placing it in a storehouse close to the track is about 3 cents per barrel. If the railroad tracks do not run to the site of the construction work, there must also be added the cost of teaming, which would amount on a distance of 1 mile to about 5 cents per barrel. In addition to this must be figured the cost of handling and returning empty sacks, the freight on same, and the loss of a few damaged or torn bags. This is usually estimated at about 2 cents per barrel. Tabulating the above, the cost of cement per barrel ready for use would appear as follows:

Cement	$0.85	
Freight	0.40	
Cloth sacks	0.40	
Total cost of cement f,o.b. cars at job	$1.65	per barrel
Deduct credit for empty sacks	0.40	
	$1.25	
Add cost of testing	0.03	
Add cost of unloading	0.03	
Add cost of teaming, if any	0.05	
Add cost of bundling and returning empty bags, and loss on same	0.02	
Net price of cement ready for use in concrete	$1.38	per barrel.

It is usual to obtain quotations from the cement companies for cement for jobs on which estimates are being made. These quotations always include the freight and the bags, and to

arrive at the net cost it is necessary to deduct for the bags and add for the supplementary items, according to the above list.

Sand usually costs about 30 cents per cubic yard to dig and load on teams or cars. If it has to be screened or washed, it will cost from 40 to 50 cents per yard. Teaming or freight will vary according to the length of haul, but will usually bring the cost of sand, ready for use, up to \$1 per yard, f. o. b. the job. If it comes by rail, there should be added to this 15 cents per yard for unloading from cars.

Crushed stone can be bought at from 85 cents to \$1 per ton at the crusher, to which must be added the cost of teaming or freight, which will vary according to the length of haul. On a haul of moderate length it is usual to pay from 20 to 40 cents per ton, so that the cost of crushed stone (f. o. b. the job ready for use) generally varies between \$1.25 and \$1.40 per ton. To this should be added about 20 cents per ton if it has to be unloaded from railroad cars. If gravel of suitable size and quality is available for use, it can generally be obtained for \$1.30 per yard, a considerable saving on the price of crushed stone. In comparing the price of gravel and crushed stone, 1 cu. ft. of crushed trap rock or granite may be considered as weighing 100 lb.

The labor of mixing and placing concrete varies considerably, according to the conditions of the job and the nature of the work. It is obvious that to mix and place concrete in heavy bridge abutments and concrete dams would cost a great deal less than to place concrete in floor slabs 3 or 4 in. thick, or in beam and column forms, as the former would not require so much spading and spreading.

Assuming a well laid-out job and a machine mixer taking four bags to the batch, the cost of mixing concrete should be from 50 to 60 cents per cubic yard. The operations will consist of loading wheelbarrows with sand and stone and wheeling them up to the mixer and charging same, bringing cement from the cement shed and putting it into the mixer, and the work of an engineer in running the mixer and discharging the concrete into wheelbarrows.

The cost of placing concrete will include the wheeling and dumping of the concrete in place, and the spreading and spading of the concrete in the forms of a building. This should cost about 75 cents per yard. In columns and thin walls, where

there is a lot of spading and where care has to be used to get a good surface on the concrete, this cost would be about doubled. On heavy masses of concrete, such as dams and thick retaining walls, these prices can be considerably reduced, especially if the plant is well laid out and the equipment is good. Concrete has been mixed and placed by mixer and derrick, or tracks and cars, for as low as 50 cents per cubic yard.

The following is an approximate schedule of labor prices for mixing and placing concrete:

Mixing and placing in footings....................	$1.50 per cu. yd.
Mixing and placing in floor slabs not exceeding 4½ in. thick....................................	1.60 per cu. yd.
Mixing and placing in floor slabs exceeding 5 in. thick.	1.00 per cu. yd.
Mixing and placing in columns and thin walls......	1.50 per cu. yd.
Mixing and placing in walls exceeding 18 in. in thickness..................................	1.00 per cu. yd.
Mixing and placing in dams and thick retaining walls....................................	0.70 per cu. yd.

On some jobs it is possible to unload sand and gravel direct from railroad cars to the wheelbarrows which charge the mixer. In such cases the materials would be handled once instead of twice before going into the mixer, and a saving of about 20 cents per yard would result.

The cost of tools and plant and supplies on a job varies a good deal according to the nature of the work. But, assuming a job containing 5000 cu. yd. of concrete work carried out by a contractor of ability, it will usually be found that the cost of plant, tools, and supplies, temporary buildings, office, cement shed, etc., will amount to about $5000. Of this amount, about $700 would be spent on labor in setting up the plant and dismantling it; $300 for freight; $1500 for small tools and depreciation of mixer, hoisting engines, and large tools; and $2000 for coal or power, also small supplies and sundries. The writer's practice is to estimate $1 for every yard of concrete on jobs containing between 4000 and 10,000 yd. of concrete. On larger jobs than this, the proportion would be smaller, probably about 70 to 80 cents; and, on smaller jobs than those containing 3000 yd., the proportion would be higher—from $1 to $1.40 per yard. On jobs containing less than 600 yd. of concrete, machine mixing is not usually economical, and in that case it will be necessary to estimate for mixing by hand, which will cost about $2 per yard more than the above prices.

To summarize, the cost of 1 cu. yd. of concrete on a job containing 5000 yd. of reinforced-concrete work in floors and columns, etc., may be estimated as follows:

Cement, 1⅔ barrels at $1.38	$2.30
Sand, ½ yard at $1.00	0.50
Stone, 1.35 tons at 1.40	1.89
Labor	1.35
Plant	1.00
Total	$7.04 per cu. yd.

It must be remembered that this cost does not include the cost of forms, steel, or finished surface.

In Art. 9 of Volume I will be found formulas giving the quantities of cement, sand, and stone required for 1 cu. yd. of concrete. It will be noted that in the above estimate these proportions are not adhered to. It must be remembered that the proportions referred to are the net quantities of material required as determined by careful experiment. The conditions on construction work do not approach those of laboratory work, and there is always a considerable waste of cement, sand, and stone. It has been found in practice that, when estimating, it is not safe to allow less than the following amounts of cement for different proportions of mix:

1 : 1½ : 3 mix	2.00 barrels per cubic yard
1 : 2 : 4 mix	1.66 barrels per cubic yard
1 : 2½ : 5 mix	1.40 barrels per cubic yard
1 : 3 : 6 mix	1.20 barrels per cubic yard

The amount of sand and stone required varies considerably according to the percentage of voids. This variation cannot be taken into account in the usual methods of estimating—as it can only be ascertained by careful tests and varies from time to time, even when the source of supply is the same—and therefore it is usual to allow ½ cu. yd. of sand per cubic yard of concrete, and 1 cu. yd. of crushed stone—figuring crushed stone to weigh 100 lb. per cubic foot.

117. Forms.—Forms should be measured by the square foot of surface (measuring all sides that touch the concrete) and priced according to the labor involved in erecting, studding, bracing, and stripping. Strictly speaking, form work is the labor of supporting wet concrete, and the material—that is, the lumber used—is only incidental to this labor; therefore it is not correct to take off the amount of lumber used and price it according to

the number of board feet thus estimated. It is the practice of some firms to estimate forms by the latter method but such a practice is misleading, not only on the theoretical ground taken above, but on the practical ground that no two contractors would use the same amount of lumber in erecting a given piece of form work. Besides, it is not possible to determine beforehand just how much lumber will be used for the same, since careful drawings of the forms are not usually available. In the writer's practice, he has always found that even though enough lumber to complete the work is ordered when a job is started, it is always necessary, on account of loss and waste of lumber, to order a great deal more before the job is finished.

The forms for a building consist principally of:

Forms to floors.
Forms to beams and girders.
Forms to columns.
Forms to walls.
Forms to footings.

Each of these should be measured separately and priced at separate and different rates.

The cost of forms to floor slabs will take into account:

1. Making up panels.
2. Setting up posts and bracing same.
3. Putting girts and ledgers on tops of the posts.
4. Laying the panels on the same.
5. Stripping the centering after the concrete has set.

The labor of making up form panels will average about 3 cents per square foot, and as these are generally used three or four times, 1 cent per square foot on the whole area is a good figure to use in estimating. The labor of erecting studs and bracing will average about 2 cents per square foot, and the cost of putting on joists and girts and laying down panels will also cost about 2 cents per square foot, giving a total labor cost of erecting forms of 5 cents per square foot. As there is 1 cent per square foot for stripping, we get a labor cost of 6 cents per square foot as a total. On a small and irregular building, of course this cost will be much higher, but for a plain, large factory a low cost such as this can often be reached. If the height from floor to ceiling is over 16 ft., this price should be increased, as the studs will have to be spliced and extra bracing will have to be put in. Two cents a square foot should therefore be added in such a case.

The cost of beam and girder forms includes the cost of the following operations:

1. Making beam bottoms and beam sides.
2. Erecting same on the posts and joists which support the floor slab.
3. Stripping.

The cost of making up will be about 4 cents per square foot, and as beam bottoms are left in longer than the slab forms, it is not safe to figure on using these more than twice, giving an average cost of 2 cents per square foot for making beam forms. Erecting the same will cost about 4 cents per square foot, and stripping 1 cent per square foot, giving a total cost of 7 cents per square foot for beam and girder forms. If beams and girders are haunched at the ends, 50 cents more should be allowed each time for the haunching.

The labor of forming columns may be sub-divided into:

1. Making up panels.
2. Erecting panels and placing yokes.
3. Plumbing.
4. Bolting.
5. Stripping.

The cost of making up panels, which are usually of 1½-in. stock, will be found to average between 4 and 5 cents per square foot. Allowing that these will be used three times, the cost per square foot of form work would be about 1½ cents. The erecting, plumbing, and bolting will be about 7 cents per square foot, and the cost of stripping about 1½ cents per square foot, giving a total labor cost of 10 cents per square foot for labor on column forms.

Columns less than 8 ft. high cost a good deal more per square foot than higher columns, owing to the fact that there is just as much time and labor spent in plumbing, erecting, and bolting up as if the column were twice as high with twice the amount of surface area.

In a similar way, the cost of erecting wall forms will be found to be about 7 cents per square foot, and the cost of footing forms about the same.

The writer's experience, based on the cost of over fifty reinforced-concrete buildings, large and small, shows that a good rule for estimating the cost of lumber, nails, oil, and wire used in the construction of forms to a factory building, is to reckon from $2.70 to $3.50 per 100 sq. ft. for these items, a good average being

$3 per 100 sq. ft. This figure is higher than that used by a good many estimators at the present day, but is based on actual experience of costs kept on a large number of jobs, and the writer is constantly proving that this figure is correct.

To summarize, the cost of forms per square foot to a reinforced-concrete building may be tabulated as follows:

Forms to floor slabs:	
Lumber, nails, oil, etc.	$0.03
Labor making panels	0.01
Labor erecting studs and bracing	0.02
Labor laying panels	0.02
Labor stripping	0.01
Total	0.09
Forms to beams and girders:	
Lumber, nails, and oil	0.03
Labor making	0.02
Labor erecting	0.04
Labor stripping	0.01
Total	0.10
Forms to columns:	
Lumber, nails, oil, etc.	0.03
Labor making panels	0.01½
Labor erecting, plumbing, and bolting	0.07
Labor stripping	0.01½
Total	0.13
Forms to footings:	
Lumber, nails, and oil	0.03
Labor making and erecting	0.06
Labor stripping	0.01
Total	0.10
Forms to walls:	
Lumber, nails, oil, etc.	0.03
Labor making	0.02
Labor erecting and plumbing	0.05
Labor stripping	0.01
Total	0.11

118. Steel.—The cost of steel reinforcement, if ordered in time to wait for delivery direct from the mill, will average about

$1.70 per 100 lb. The freight rate on a haul of about 500 miles will be about 18 cents, giving a total of $1.88 per 100 lb. for steel bars, f. o. b. cars at the job. The cost of steel bars varies according to the size of the bar. Bars of from 1½-in. to ¾-in. diameter are taken at the lowest rate, which is called the *base* price. Small bars take a higher rate as follows:

⅝ in. and $\frac{11}{16}$ in.	Base plus 10 cents
½ in. and $\frac{9}{16}$ in.	Base plus 20 cents
⅜ in. and $\frac{7}{16}$ in.	Base plus 50 cents
¼ in. and $\frac{5}{16}$ in.	Base plus $1.00

This *differential* rate is often halved in mill orders but stock deliveries take the full extras. This differential is an important factor in design as well as in estimating. For example, assume a floor 200 ft. × 100 ft. having a slab 6 in. thick and an area of steel per square foot of 0.462 sq. in. The total weight of steel required (allowing for laps) would be: $200 \times 106 \times 0.462 \times 3.4 =$ 33,301 lb.

Cost of ½-in. square bars 6½ in. on centers, 33,301 lb. @ $1.90 per 100 lb. = $632.70.

Cost of ⅜-in. round bars 3 in. on centers, 33,310 lb. @ $2.20 per 100 lb. = $726.00.

Thus there is a difference of $93.30 in favor of ½-in. bars if taken from stock, or $46.65 if taken from mill deliveries. The price of reinforcing bars taken from stock is of course much higher, but it is usual in estimating on a building to figure on taking the steel for the footings and the lower part of the building, enough for the first five or six weeks' work, out of stock and buy the rest direct from the mill.

The cost of unloading steel and piling it on the job is about 40 cents per ton, and, if it has to be teamed from the freight yards to the site of the work, this will cost 20 cents per ton and upward.

The cost of bending and placing steel in the building will vary according to the amount of work that is done. Thus, placing steel bars ⅝-in. diameter in a floor slab will cost about $5 per ton. If the bars have to be bent up at the end, $2 per ton should be added. Bending and placing steel bars and stirrups in beams will cost from $8 to $10 per ton. Wiring up and placing steel bars in columns and placing hoops around them will cost from $10.00 to $12.00 per ton. Placing steel of ½-in. and ⅜-in. diameter in walls will cost from $12 to $15 per ton.

These prices are sufficient to include the cost of wire, tools for bending, etc. Ten dollars per ton is a good average price for labor on steel reinforcement all through the job.

119. Surface Finish.—One hundred square feet of granolithic finish laid 1 in. thick in the proportion of one of cement, one of sand, and one of fine crushed stone, will require one barrel of cement, 4 cu. ft. of sand, and 4 cu. ft. of crushed stone. This may therefore be estimated as follows:

1 bbl. cement	@$1.38 per bbl.	$1.38
4 cu. ft. sand	@$1.00 per yd.	0.15
400 lb. fine crushed stone	@$2.00 per ton.	0.40
		$1.93
Labor mixing and placing		0.60
Finishers' time trowelling surface		1.00
Total cost per 100 sq. ft.		$3.53 or 3½ cents per square foot.

If the surface of the concrete has to be cleaned off with acid or sand blasting, this will cost from 2 to 3 cents per square foot additional.

The exterior surfaces of concrete columns and beams are frequently rubbed smooth with carborundum stone, using a little water and cement. The cost of this work, including hanging swing stages for the finishers, will be from 4 to 6 cents per square foot.

For ornamental effect, external surfaces are sometimes picked with a pointed tool or crandall hammer. The cost of this runs between 6 and 8 cents per square foot.

CHAPTER XXVI

ESTIMATING QUANTITIES

The operation of estimating quantities is that of calculating (from plans supplied) quantities of labor and material which go to make up the completed building. This is usually called *taking off* or *scaling*. This should be done quite independently of the pricing or the arithmetical work of extending the quantities to obtain the totals of the quantities of work. The secret of accurate, speedy taking off is to be found in a systematic way of going about the work. No printed forms, tables, or special rules for taking off will insure against error, but the surest way of making an accurate estimate is to have a good system to work on and a clear and easily followed way of setting down the items. The writer's practice in taking off is to use plain squared paper 8 in. × 10½ in., ruled in nine columns. Across the top of the sheet is written the name and description of the job, name of architect, etc. In the first column is placed a description of the items measured; the next four columns are for the number, length, width, and height of the members of the building; the next two are for arithmetical calculations and totals; and the last two for unit and total price. (See Fig. 410.) It is very important to keep length, breadth, and height in the same order in every item. Each can then be readily identified. In taking off a reinforced-concrete building, start with the structural members in the order in which they are built; that is, first take concrete footings, then columns, then floor slabs, then beams and girders, then curtain walls and partitions, then cornice, and then stairs and landings. Take all the concrete first, one item at a time and complete it. When all the concrete is taken off, proceed to take off the forms, and after that take off the reinforcement, and then the finish to the surfaces. After that take off excavation, windows and doors, roofing and other incidental items necessary to complete the cost of the building.

In putting down the dimensions, it is well to put a note identifying each item, thus:

Concrete columns:

Basement, (mark A)......$5 \times 1\frac{1}{2} \times 1\frac{1}{2} \times 10$
(mark B)......$4 \times 1\frac{1}{2} \times 1\frac{2}{3} \times 10$
(mark C)......$7 \times 1\frac{1}{2} \times 1\frac{1}{3} \times 10$
First floor,(mark) A......$5 \times 1\frac{1}{3} \times 1\frac{1}{3} \times 10$
(mark B)......$4 \times 1\frac{1}{3} \times 1\frac{1}{2} \times 10$
(mark C)......$7 \times 1\frac{1}{3} \times 1\frac{1}{3} \times 10$

It takes a little more time to do this, but it is well worth the labor, and any item can be readily identified afterwards. Also if an item is left out in error, it can be more easily detected. It

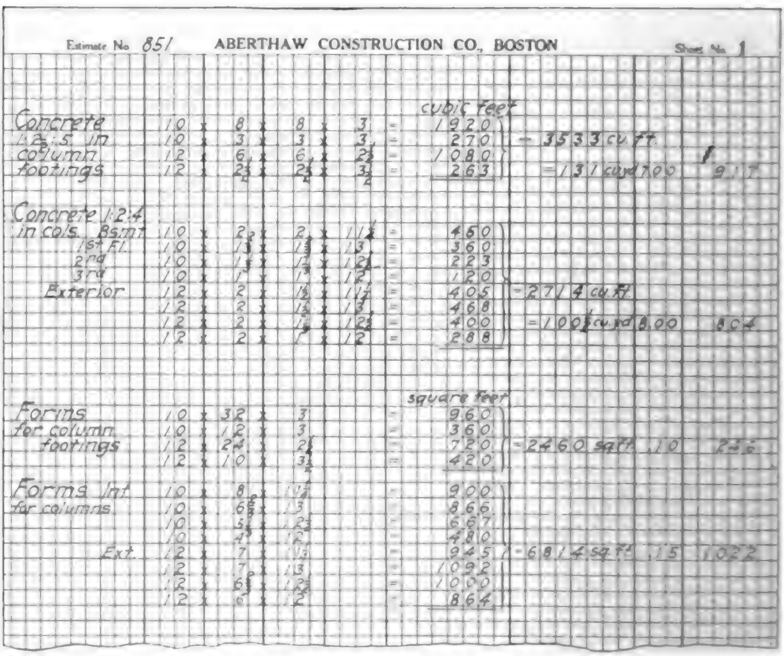

FIG. 410.

is the writer's practice to put all dimensions in feet and fractions. Some estimators work in feet and inches and some in feet and decimals. There seems to be the least chance for error in using fractions, but this is a matter of individual judgment.

120. Concrete.—The following rules should govern the measurement of concrete work:

All concrete should be measured by the cubic foot or cubic

yard, and in all cases forms should be measured separately. All concrete should be measured net as placed or poured in the structure or building, and an excess measurement of concrete should never be taken to pay for the cost of forms or extra labor in placing. All openings and voids in concrete should be deducted, but no deduction should be made for steel reinforcement, I-beams, bolts, etc., embedded in the concrete, except where such have a sectional area of more than 1 sq. ft. No deduction should be made for chamfered, beveled, or splayed angles to columns, beams, and other work.

For beams and girders it is usual to show on the plan the depth of concrete from the top of the slab. Thus, if the quantity of concrete for the slab has been taken right through, it will be necessary to consider only the extra concrete below the slab in taking off beam and girder quantities. For example, in a floor 6 in. thick having 12-in.×30-in. girders, the concrete to take off for the girders should be considered as 1 ft. wide by 2 ft. deep, since the other 6 in. is included in the slab.

Each class of concrete having a different proportion of cement, sand, or aggregate should be measured and described separately. Concrete in the different members of a building or structure should be measured and described separately according to the accessibility, location, or purpose of the work; concrete in floor slabs should be measured and priced separately from columns or walls, and so on. Concrete with large stones and rocks embedded in same (cyclopean masonry) should be measured as one item and described according to the richness of the mix and the percentage of rock in same. Concrete in stairs should be measured by the cubic foot, and it is usual to include surface finish with same in this case, as it forms such a small item in the cost.

121. Forms.—Forms should be measured in square feet, taking the area of the surface of the concrete which is actually touched by the forms or false work. Forms should in all cases be measured and described as a separate item and never included with the concrete. No deduction should be made in measurement of surface of concrete supported by forms because of forms being taken down and re-used two or three times in the course of construction.

It is not necessary to consider struts, posts, bracing, bolts, wire ties, oiling, cleaning, and repairing forms, as these should be covered by the price put on the square foot measurements.

Forms to the different parts of a building should be measured and described separately according to their nature; that is, forms to floor slabs, walls, columns, footings, etc., should be separated from each other. No allowance need be made for angle fillets or bevels to beams and columns, etc., but curved mouldings should be measured and described separately.

No deduction in measurement of forms should be made for openings having an area of less than 25 sq. ft. as the labor in forming same is often greater than the cost of the omitted area. No deduction should be made in floor forms for heads of columns, or in column and girder forms for ends of girders, cross beams, etc. No allowance should be made for pockets in column forms for clearing out rubbish.

The correct measurement of column forms is the girth of the four sides, or circumference, multiplied by the height from the floor surface to the under side of floor slab above. Forms to

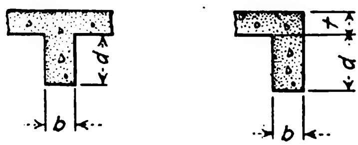

FIG. 411.

octagonal, hexagonal, and circular columns should be measured and priced separately from forms to square columns. Caps and bases to columns and other ornamental work should be enumerated and fully described by sketches in the estimate with overall dimensions.

The correct measurement of beam forms is the net length between columns multiplied by the sum of the breadth (b) and twice the depth below the slab (d), except for beams at edge of floor or around openings, which shall have the thickness of the floor (t) added to the sum of the breadth and twice the depth. (See Fig. 411.)

Wall forms should be measured for both sides of concrete walls.

Forms to the upper side of sloping slabs such as saw-tooth roofs should be measured whenever the slope of such slab with the horizontal exceeds an angle of 25 degrees.

Mouldings in form work should be measured by the linear

foot. Forms to circular work should always be measured separately from forms to straight work.

No measurement or allowance should be made for construction joints in slabs, beams, etc., to stop the day's concreting, but construction joints or expansion joints in dams and other large masses of concrete should be measured by the square foot as they occur.

Forms to cornices should be measured by the linear foot and the girth stated. Plain forms to back of cornice should be measured separately. Forms to window sills, copings, and similar work should be measured by the linear foot. Forms to the underside of stairs should be measured by the superficial foot, and forms to the front edge by the linear foot. Forms to the ends of steps should be measured by number.

122. Steel.—Reinforcing bars should be measured by the linear foot and reduced to weight in pounds for pricing. The net weight placed in the building should be taken and no allowance made for waste and cutting, or wire ties and spacers, etc., but laps should be allowed for as called for by the plans or by the necessities of the design. Deformed bars should be measured separately from plain.

The cost of bending and placing in columns and beams is greater than in slabs, but as the difference is not great it is not usual to make any distinctions but to take off the whole of the steel together, except in special cases.

Pipe sleeves, turnbuckles, clamps, threaded ends, nuts, forgings, and other special items should be measured separately by number and size, and allowed for in addition to the weight. Wire cloth, expanded metal, and other steel fabrics sold in sheets are measured and described by the square foot. The size of mesh and weight per square foot of steel will govern the price, and should be stated. All laps should be measured and allowed for.

123. Surface Finish.—Finish of concrete surfaces should be measured by the square foot. Finish should always be measured and described separately. No measurement or allowance should be made for going over concrete work after removal of forms, and patching up voids and stone pockets, removing fins, etc., as this is part of the labor incidental to placing the concrete and the cost will depend upon the care used in spading the concrete into the forms.

Granolithic finish should be measured by the square foot and should include all labor and materials for the thickness specified. Finish laid integral with the slab should be measured separately from finish laid after the slab has set. No allowance should be made for protection of finish with sawdust, sand, or covering in to protect from weather. Grooved surfaces, gutters, curbing, etc., should be measured separately from plain granolithic and should be measured by the square foot or linear foot, as the case may require.

Putting on cement wash, rubbing with carborundum, scrubbing with wire brushes, tooling, and picking, are other surface labors that should each be separately measured and priced. The price should include the use of swing stages, tools, and materials required.

CHAPTER XXVII

EXAMPLE OF AN ESTIMATE FOR A CONCRETE BUILDING

The sheets which follow give an actual estimate for the reinforced-concrete work of the concrete building for the Davis Fish Company, shown on Plates XXIV to XXXIV inclusive. The method of *taking off* is sufficiently described in Chapter XXVI and needs no further explanation here.

A few notes regarding local conditions are needed to make the pricing clear.

An inspection of Plate XXXII will show that the building is on a small lot with a main street in front, a private way on one side, an adjacent building on the other side, and a small wharf and dock at the back. The contractor could figure on using the wharf for depositing materials, and also part of the main street, but could not use the private way, and as the street is the main street of the town he would have to be careful not to obstruct traffic with his materials. The railroad is a mile and a half away and the teaming from the railroad depot to the job is all down hill.

Cement was quoted \$1.65 f.o.b. cars at Gloucester, including bags. Deducting \$0.40 for bags gives \$1.25 net. Add \$0.03 for testing, \$0.07 for unloading from cars and teaming to job, and \$0.03 for unloading from teams and placing in storehouse. Add also \$0.02 for bundling and returning empty bags and loss on damaged bags. This will give a net price of \$1.40 per barrel for Portland cement ready for use in the concrete.

The sand was quoted at \$1.50 per cubic yard by local dealers, delivered by water f.o.b. the dock.

Crushed stone was quoted at \$1.80 per ton f.o.b. teams at the job by a local quarry owner whose quarry was situated 2 miles away, the haul being half uphill and half down hill.

The labor cost of mixing and placing concrete was estimated to average \$1.50 per cubic yard, as the small lot and crowded street would interfere with economical work.

1

– Estimate –

– for New Manufacturing + Office Bldg –

at Gloucester, Mass.

for The Frank E. Davis Fish Co.

Densmore + Le Clear, Engineers

Mar. 14th 1911

Concrete.

Footings 1:2½:5

Corners	4 × 5½ × 5½ × 1½	182				
	4 × 4¼ × 4¼ × 1½	108				
S. E. + W.	10 × 5 × 4 × 1½	300				
	10 × 3¾ × 2¾ × 1½	154				
West	3 × 5½ × 3 × 1½	74				
	3 × 4¾ × 2¼ × 1½	48	= 2010 cf			
North	4 × 5¼ × 4 × 1½	126				
	4 × 4¼ × 2¾ × 1½	70	= 75 cu yd.			
North	17 × 4½ × 1½	115				
	16 × 3½ × 1½	84		7.80	585	
Interior	12 × 5 × 4¾ × 1½	428				
	12 × 3½ × 3¼ × 1½	205				
	3 × 2 × 2 × 1½	18				
	3 × 1½ × 1½ × 1½	10				
Col. Slabs N.	6 × 3 × 1⅓ × 1¼	30				
West	2 × 3½ × 1⅓ × 3¼	30				
	(3×3 + 2×8) × 1⅓ × 1¼	28				

Concrete Walls. 1:2:4

North	(3 + 84⅔) × 1⅓ × 10½		1227			
Xtra at Ent.	14⅔ × 1⅓ × 1¼		24			
" " "	12 × ⅔ × 10		80	= 2335 c.f		
Windows.	6 × 8½ × 1⅓ × 1¾	ddt 119		319 ddt		
West	61¼ × 1⅓ × 10½		857	2016		
Windows.	2 × 6¼ × 1⅓ × 2	ddt 33		= 75 cu yd	8.18	614
"	3 × 6¼ × 1⅓ × 4½	" 112				
Door	6½ × 1⅓ × 7½	" 55				
W. Beam Sa.	7 × 9½ × [illegible] × 1½		79			
[illegible] for Ent.	(17 + 12½ + 3½ + 17) × $\frac{17}{24}$ × 1¼		68			
						1199

Ruled form is shown in Fig. 410.

2

Concrete Columns, etc. 1:2:4

N. Entrance	2	× (3⅝×1+2⅝×1)× 12	152		
Arch Xtra.		(6⅝×5−19°) × 2¼	32		
1st flr.	4	× (2⅛×1+2×⅝×1/12)× 10¼	91		
2nd + 3rd	12	× (2⅛×1+2×⅝×1/12)× 21	556		
N corners 1st, 2nd & 3rd	2	× (3⅛×1+2×⅝×1/12)× 31⅙	237		
E & W 1st, 2nd + 3rd	3	× (2⅛×1+2×⅝×1/12)× 31⅙	208		
E. Bas't		(2⅛×1+2×⅝×1/12)× 13	29	3596 c.f	
E. Chimney		(2⅝ × 2½ − 1⅝ ×1)×(13+31⅙)	265		
South	6	× (2⅛×1+2×⅝×1/12)×(15+31⅙)	616	= 133 cu.yd 8.18	1088
" corners	2	× (3⅛×1+2×⅝×1/12)× 31⅙	237		
bas't.		(3⅛×1+2×⅝×1/12)× 15	52		
int. bas't.	12	× 1⅝ × 1½ × 12½	412		
1+2	12	× 1½ × 1½ × 20½	554		
3	11	× 1 × 1 × 10⅝	118		
	1	× 1¼ × 1 × 10⅝	13		
	3	× 1 × 1 × 8	24		

Concrete floor slabs & beams. 1:2:4

Slab. 1st	1	× 82⅛ × 59 × ½	2429		
2nd & 3rd	2	× 82⅛ × 59 × ⅔	6477		
roof.	1	× 82⅛ × 59 × 5/12	2024		
1st		(6½×5½+12½×4½) × ½ ded. 46			
2nd + 3rd		2×(9×10+6½×5½)×⅔ ded. 167			
roof.		17 × 10 × 5/12 ded. 70			
Beams. 1st	12	× 18 7/12 × 1½ × 1¼	418		
	6	× 18⅛ × 1½ × 1¼	205		
A		10⅛ × ½ × 1	5		
B		5⅝ × ½ × ⅓	1		
X		8½ × ½ × 5/12	2	= 14809 c.f.	
Y		10⅛ × 1 × ⅚	9	283 ded.	
" 2nd & 3rd	2×12	× 18¾ × 1½ × 1¼	844	14526	
	2×6	× 18½ × 1½ × 1¼	416		
Y		10⅛ × ⅞ × ½	4	= 538 cu.y 8.18	4401
" roof	12	× 19 × ⅚ × 1¼	238		
	6	× 19 × ⅚ × 1¼	119		
14–18, 14½–18½	2	× 10⅜ × ⅝ × 1 1/16	20		
Y		10⅛ × ½ × 7/24	4		
Wall Beams 1st, E & W		(84⅜+61−12)×1½× 1¼	233		
S	7	× 9½ × 1 × 1¼	115		
E		(2 × 17+17⅚)× 1 × 1¼	91		
2nd + 3rd, E & W		(4 × 17½+2×17⅚)×11/12× 1¼	167		
N + S.	27	× 9½ × 5/12 × 1¼	410		
Roof, E.		(2 × 17½+17⅚)×1 × 2¼	117		
S	7	× 9½ × 1 × 2¼	150		
N.	7	× 9½ × 1½ × 2¼	175		
W.		(2×17½ + 17⅚)×1½× 2¼	136		

5489

3

Concrete Curtain Walls 1:2:4

E. Basm't	$(2 \times 17\frac{1}{2} + 17\frac{5}{8}) \times \frac{3}{4} \times 8\frac{1}{2}$		295		
Door	$3\frac{1}{2} \times \frac{3}{4} \times 7$ ddt 16				
Window	$8\frac{1}{2} \times \frac{3}{4} \times 3$ " 17				
S. Basm't	$4 \times 9\frac{1}{2} \times \frac{1}{2} \times 4\frac{1}{2}$		86	= 1350 c.f.	
N. 1st Flr.	$6 \times 9\frac{1}{2} \times \frac{1}{2} \times 3\frac{1}{4}$		92	33 ddt	
window 2nd	$12 \times 2\frac{1}{2} \times 3\frac{1}{4}$		98	1317 c.f	
S	$7 \times 9\frac{1}{2} \times \frac{1}{2} \times 3\frac{1}{4}$		108		
W.	$(2 \times 17\frac{1}{2} + 17\frac{5}{8}) \times \frac{1}{2} \times 3\frac{1}{4}$		85	= 49 cy 8.10	401
N & S 2nd Flr.	$13 \times 9\frac{1}{2} \times \frac{1}{2} \times 3\frac{1}{4}$		200		
W	$52 \times \frac{1}{2} \times 3\frac{1}{4}$		85		
N & S. 3rd Flr.	$14 \times 9\frac{1}{2} \times \frac{1}{2} \times 3\frac{1}{4}$		216		
W	$52 \times \frac{1}{2} \times 3\frac{1}{4}$		85		

Concrete Cornice & Parapet &c 1:2:4

Cornice N & W.	$156 \times 2\frac{1}{4} \times 2\frac{1}{2}$	878	= 1313 c.f	
parapet	$(2\frac{1}{4} + 156) \times \frac{3}{4} \times 2\frac{1}{4}$	267		
S & E.	$138 \times \frac{3}{4} \times 1\frac{1}{4}$	129	= 49 cy 8.18	401
Chimney above roof	$(2\frac{3}{4} \times 2\frac{1}{2} - 1\frac{1}{3} \times 1) \times 7$	39		

Concrete Paving 1:2½:5

	$82\frac{1}{2} \times 59 \times \frac{5}{12}$	2028	= 2028	
Elevator	$6\frac{1}{2} \times 5\frac{1}{2} \times \frac{5}{12}$ ddt 15		30 ddt	
Pipe pit	$1 \times 1 \times \frac{5}{12}$ " –		1998	
Trench	$29\frac{1}{2} \times 1\frac{1}{4} \times \frac{5}{12}$ " 15		= 74 cy 7.80	577

Concrete Stairs

incl'g steel, forms	$4 \times 7\frac{1}{2}$	30		
and finish	15×4	60	90' lin 1.25	113

Concrete in elevator & plumbing pit trench &c 1:2:4

Walls Elev pit	$26 \times \frac{1}{2} \times 3$	39		
Paving " "	$6\frac{1}{2} \times 5\frac{1}{2} \times \frac{1}{3}$	12		
Walls 1x1 pit	$5 \times \frac{1}{3} \times 1\frac{1}{2}$	2	= 90 c.f	
Paving	$1 \times 1 \times \frac{1}{3}$	–		
Walls Trench	$2 \times 29\frac{1}{2} \times \frac{1}{3} \times 1\frac{1}{4}$	25	= 3 cu yd 8.18	25
Paving "	$29\frac{1}{2} \times 1\frac{1}{4} \times \frac{1}{3}$	12		

Forms to footings

$4 \times (22+17) \times 1\frac{1}{2}$	234		
$10 \times (18+13) \times 1\frac{1}{2}$	465		
$3 \times (17+14) \times 1\frac{1}{2}$	140		
$4 \times (18\frac{1}{2}+14) \times 1\frac{1}{2}$	195		
$(43+39) \times 1\frac{1}{2}$	123	1988 sq ft .12	239
$12 \times (19\frac{1}{2}+13\frac{1}{2}) \times 1\frac{1}{2}$	594		
$3 \times (8+6) \times 1\frac{1}{2}$	63		
$6 \times 8\frac{2}{3} \times 1\frac{1}{4}$	65		
$2 \times 9\frac{2}{3} \times 3\frac{1}{4}$	63		
$36\frac{2}{3} \times 1\frac{1}{4}$	46		

1756

4

Item	Dimensions	Sq. ft.	Total	Rate	Amount
Forms to Basement Walls		Sq. ft.			
	2 × 87⅓ × 10½	1841			
	29½ × 1¼	36			
	13⅓ × 10	133	3677 sq ft	.12	441
	2 × 61⅓ × 10½	1286			
Wall Bm's Bsmt	7×2 × 9½ × 1½	200			
Pier	2 × (2×17+17⅓)×1¾	181			
Forms to St. and	2 × (7½+4) × 12	272			
arch	2 × (6⅓×5−19) × 2	58	355 sq ft	.40	142
" soffit	11 × 2¼	25			
Forms to Colums					
N. 1st	4 × 6⅔ × 10¼	274			
2nd + 3	12 × 6⅔ × 21	1680			
corners	2 × 9⅔ × 31⅙	613			
E + W 1, 2 + 3	3 × 6⅔ × 31⅙	623			
E. Bas't	6⅔ × 13	86			
" stg	10⅔ × (13+31⅙+7)	532			
S.	6 × 6⅔ × 46⅙	1847			
corners	2 × 9 × 31⅙	613	9504 sq ft	.13	1236
S.E. Bas't	9⅔ × 15	147			
Int.	12 × 6⅔ × 12½	1000			
1 + 2	12 × 6 × 20½	1476			
3	11 × 4 × 10⅔	469			
	1 × 4½ × 10⅔	48			
	3 × 4 × 8	96			
Forms to floor slabs 1st	84⅚ × 61½	5216			
2nd + 3rd	2 × 84⅓ × 61	10288			
roof	84⅓ × 61	5144			
ells	3 × 6½ × 5½ ddt 107				
	(12½×4½ + 2×9×10) " 236				
	17 × 10 " 170				
Beams & girders (sides only, (soffits flush with slabs)	12×2× 18⅔ × 1¼	558			
	6×2× 18⅙ × 1¼	273			
	2 × 10⅙ × 1	20			
	2 × 5⅙ × ⅓	4			
	2 × 8½ × 5/12	7	26936		
	2 × 10⅙ × ⅚	17	513 ddt		
	24×2× 18¾ × 1¼	1125	26423 sq ft	.10	2642
	12×2× 18½ × 1¼	555			
	2 × 10⅓ × ½	10			
	12×2× 19 × 1¼	570			
	6×2× 19 × 1¼	285			
	2×2× 10⅔ × 1⅖	60			
	2 × 10⅓ × 19/24	17			
Wall Beams	133 × (1¾+1¼)	399			
	66½ × 3	200			
	51⅙ × 3	154			
	(68⅙+35⅙) × 3	312			
	27 × 9½ × 3	770			
	52 × 4	208			
	67½ × 4	270			
	(66½+52) × 4	474			
					4461

5

Extra on wall beam form for molded projection forming First Floor belt course		120 lin ft	.20	24
Forms to Curtain Walls	sq ft			
2 × 52 × 8½	884			
2 × 38 × 4½	342			
2 × 57 × 3¼	370			
2 × 66½ × 3¼	432			
2 × 52 × 3¼	338	4709 sq ft	.11	518
2 × 123½ × 3¼	803			
2 × 52 × 3¼	338			
2 × 133 × 3¼	864			
2 × 52 × 3¼	338			
Forms to Parapet + Back of Cornice				
156 × 7	1092	1437 sq ft	.11	158
138 × 2½	345			
Forms to face of molded cornice 3'-0" girth.		156 lin ft	.90	140
Forms to Pit &c				
Walls. deep pit 2 × 26 × 3	156			
" pipe " 2 × 5 × 1½	15	319 sq ft	.12	38
" trench 4 × 29½ × 1¼	148			
Extra for molds + casting caps + bases of 13 columns				125

Steel Reinforcement

all plain round bars	ft. lin.
Columns. ⅞" 4 × 8 × (15+13)	896
16 × 6 × (16+13)	2784
1¼" 12 × 4 × (15+13)	1344
½" 3 × 4 × 8	96
2nd F. ⅞" 4 × 8 × 13	416
⅞" 16 × 6 × 13	1248
1" 12 × 4 × 13	624
3rd ⅞" 4 × 6 × 11	264
⅞" 16 × 6 × 11	1056
⅞" 12 × 4 × 11	528
¼" 4×4× 13 × 10½	2184
4×16× 13 × 7½	6240
4×12× 13 × 6	3744
3 × 8 × 4	96
Girders 1st N&S ⅞" 14 × 2 × 14	392
W ⅞" 3 × 2 × 22	132
E. 1" 3 × 2 × 28	168
⅞" 3 × 3 × 23	207
½" 3 × 10 × 6	180

collected on sheet 7 — —

1003

6

Steel Reinforcement contd.

	Size	Quantities	Lin. ft.
Int.	1½"	16 × 2 × 30	960
	1½"	16 × 3 × 24	1152
	½"	18 × 10 × 6	1080
14-15, 18-19	1"	2 × 2 × 30	120
	½"	2 × 3 × 23	138
A	¾"	2 × 11½	23
B	¾"	2 × 8	16
	¾"	4 × 2½	10
X	¾"	2 × 11	22
Y	½"	2 × 12½	25
	½"	1 × 12	12
	¾"	4 × 2½	10
2nd & 3rd N & S.	½"	27 × 2 × 14	756
E & W.	1"	12 × 2 × 28	672
	1"	12 × 1 × 24	288
	¾"	12 × 2 × 23	552
	½"	12 × 5 × 6	360
Int.	1½"	32 × 2 × 30	1920
	1½"	32 × 2 × 24	2304
	½"	32 × 20 × 6	3840
14-15	1½"	2 × 2 × 30	120
	¾"	2 × 3 × 23	138
	½"	2 × 6 × 6'	72
18-29	1½"	2 × 2 × 30	120
	¾"	2 × 3 × 23	138
	½"	2 × 6 × 6	72
16-20	½"	3 × 15	45
roof. N	½"	7 × 3 × 15	315
S.	¾"	7 × 2 × 15	210
E.	¾"	3 × 3 × 23	207
	¾"	2 × 3 × 10	60
	¾"	3 × 8 × 7	168
W.	¾"	3 × 3 × 23	207
	¾"	2 × 3 × 10	60
Int.	1"	18 × 2 × 28	1008
	¾"	18 × 3 × 27	1458
	¾"	18 × 12 × 5	1080
14-18, 14½-18½	½"	2 × 2 × 13	52
	¾"	2 × 6 × 5	60
Floor beam X & Y	½"	2 × 2 × 12½	50
	½"	2 × 1 × 12	24
	¾"	2 × 4 × 2½	20
Y roof.	½"	2 × 11	22
slab 1st	½"	12/5½ × 61 × (84+2)	10975
2 & 3.	¾"	12/6½ × 61 × (84+2)	10020
roof.	½"	12/7 × 61 × (84+2)	8993
1, 2, & 3.	½"	3 × 12/7 × 84 × (61+2)	10584
truss	½"	⅔ × 36 × 30	720
cornice per.	¾"	3 × (156+12)	504
N & W.	½"	18 × (156+8)	2952
	½"	78 × (1½+3½)	390
	¾"	78 × 3	234
S & E.	½"	4 × (138+6)	576
Waterfall N & W	½"	7 × (150+8)	1106

collected on sheet 7 — —

7

Steel Reinforcement Cont'd

			lin. ft.
Ft. Wall 5/8"	10	× (84 + 4)	880
1/2"	84	× 7	588
	8	× (84 + 4)	704
	42	× 7	294
Ft. Cont. Beams	3	× 14	42
1/2"	9	× 12	108
Curtain Wall 1/2"	8	× (61 + 2)	504
	31	× 8	248
E 1/2"	9	× (61 + 2)	567
	31	× 8	248
S	5	× 38	190
	20	× 4½	90
1st N	3	× 57	171
	30	× 3	90
S	3	× 67½	202
	35	× 3	105
	3	× 52	156
	27	× 3	81
2nd N + S	3	× 124	372
	65	× 3	195
W	3	× 52	156
	27	× 3	81
3rd N + S	3	× 133	399
	70	× 3	210
W	3	× 52	156
	27	× 3	81
Chy above roof 1/2"	6	× 7	42

collected below

Steel Collected

	lin. ft.			lb				
1½"	1920	@	6.008 =	11535	Base 30½ tons		32.60	994
1⅜"	960	@	5.049 =	4847				
1¼"	1464	@	4.173 =	6109				
1⅛"	3576	@	3.379 =	12083				
1"	2880	@	2.67 =	7690				
7/8"	7211	@	2.044 =	14739				
3/4"	2289	@	1.502 =	3438				
5/8"	11514	@	1.043 =	12009	=	6	@ 33.60	201
1/2"	39826	@	.667 =	26564	=	13¼	@ 34.60	458
3/8"	1371	@	.375 =	514	=	¼	@ 37.60	9
1/4"	23568	@	.167 =	3936	=	2	@ 42.60	85

Extra cost of steel taken from stock	10	@ 10.00	100
Unloading & teaming, bending & placing steel reinforcement.	52	@ 11.00	572
			2419

8

Extra cost of Grano. finish
(thickness measured with floor concrete)

2 × 84⅓ × 61½	10414	= 20975		
2 × 84⅓ × 61	10288	343 ddt		
elev 3 × 6½ × 5½ ddt 107		20632 sq ft	.02	413
12½ × 4½ + 2 × 9 × 10 " 236				
cornice 156 × 1¾	273			

Sanitary base (cleaning room) 4" high 132' lin — 132 lin ft .20 — 26

Rubbed + cement wash surfaces

Cols. N Elev. 1st Fl. 6 × 3 × 11	198			
2 × 1⅓ × 11	29			
2nd + 3rd 16 × 3 × 10	480			
bas't 7 × 4 × 2½	70			
Wall B.N 1st Fl. 72 × 2	144			
2nd 3rd 13 × 9½ × 2	247			
roof 84⅓ × 2½	211			
Curtain Walls 1st 6 × 9½ × 3	171			
2 + 3 13 9½ × 3	370			
Cornice N + W 156 × 3	468	. 6977		
Parapet. N + W 156 × 2¼	351	100 ddt		
Cornice porch 12 × 4	48	6877 sq ft	.06	413
Column West 4 × 3 × 31	372			
Walls N beam 1st 6¼ × 8½	520			
" 2 + 3 6 × 17⅓ × 2	208			
roof 61¼ × 2½	153			
Curtain walls 9 × 17⅓ × 3	468			
Window walls 100⁰ ddt 100				
Cols South 8 × 3 × 41	984			
Wall Beams 2/3 21 × 9½ × 2	400			
roof 84⅓ × 2½	211			
Curtain walls 4 × 9½ × 4½	171			
21 × 9½ × 3	598			
Parapet S. 84⅓ × 1¼	105			

Picked Surface rubbed draft lines,
and wash to Front Entrance.

2 × 5⅓ × 10	164⁰'	.25	41
14 × 5-19			

Hydrated lime for waterproofing walls + cornice &c — 6 Tons 15.⁰⁰ — 90

Cinder Concrete Cricket-roof

(84⅓ × 59 − 17 × 10) × ¼ — 1201 c.f ∴ 45 cu yds 4.⁰⁰ — 180

1163

9

Flue linings	50 f.l.	.50	25
Load tests as specified			25
Contingencies & Sundries.			500

Totals

sheet	1	1199
	2	5489
	3	1756
	4	4461
	5	1003
	6	—
	7	2419
	8	1163
	9	550

Total net cost exclusive of contractor's profit } $18040

The concrete plant was estimated to cost about $1250 and, as there are about 1000 cu. yd. of concrete in the job, this is carried in the estimate at $1.25 per cubic yard.

To summarize, therefore, the cost of 1:2:4 concrete would show as follows:

1⅔ bbl. cement @ $1.40	$2.34
½ yd. sand....... $1.50	0.75
1.3 tons crushed stone @ $1.80	2.34
Labor mixing and placing	1.50
Plant	1.25
Total, per cubic yard for concrete in place	$8.18

1:2½:5 concrete would cost the same except that 1.4 barrels of cement only would be required, at $1.40=$1.96, a difference of $0.38 per cubic yard, making the cost of 1:2½:5 concrete $7.80 per cubic yard.

Forms to footings.—The footing forms have sloping tops and would cost more than square footing forms. These are therefore priced at $0.12 per square foot.

Forms to basement walls.—These vary in height and are done in small pieces and are priced, therefore, at $0.12 per square foot.

Forms to front entrance.—Under this heading is included 355 sq. ft. of form work with rustic jambs, arch, cornice, and pediment. The cost of same is entirely a matter of judgment and is estimated at $140, which is equal to $0.40 per square foot.

Forms to Columns.—These are of ordinary construction and are estimated at $0.13 per square foot.

Forms to floor slabs.—It will be noted that the basement story is 3 in. less in height than the first and the second stories, so that the posts will have to be spliced or wedged when used in the first story. Splicing or wedging of posts will also be needed under the roof where the ceiling is 5 in. higher than in the second story. A fair average cost of forms to the floor slabs would be $0.10 per square foot.

Forms to beams and girders.—These will be of the same size—except in the roof, where they are deeper and narrower and longer, and will require re-making. An average price of $0.10 per square foot is estimated for these.

Forms to parapet and back of cornice.—These are simple wall forms and are priced at $0.11 per square foot.

Forms to face of moulded cornice.—These forms would be specially built up and would require some specially dressed stock. This form work should cost about \$0.30 per square foot, or \$0.90 per linear foot.

Moulded caps and bases to front columns, shown on Plates XXIV and XXV.—These would be first cast in concrete and then set into the forms and the column concrete poured around. Counting the corners as two each, there will be thirteen of these caps and thirteen bases. They will require two moulds for the caps and two for the bases. The corner ones would be made and cast first and the moulds then altered to do the others—the two bases by the front door being cast last. The cost of this work should be about \$125.

Steel reinforcement.—The base price of steel bars at the mill at this time was \$1.45 per 100 lb. Adding \$0.18 freight rate gives \$1.63 per 100 lb. for the steel f.o.b. cars at Gloucester, equal to \$32.60 per ton. This job would require about 10 tons, to be taken from stock at an advance of \$10 per ton. The cost of unloading from cars and teaming to job would be \$0.80 per ton, the cost of unloading and piling on the job \$0.50 per ton. For bending and placing, an average price would be \$9 per ton; the cost of wire and sundries, \$0.50 per ton; giving a total of \$10.80 per ton for labor and teaming, or say \$11 per ton.

Granolithic finish.—The full thickness of the concrete has been measured with the slab, so the price to be placed on the granolithic finish will be the extra value only. The cost of finish as per Chapter XXV is \$0.03½ per square foot. The cost of 1 in. of concrete already estimated at \$8.18 per cubic yard, equals \$0.30 per cubic foot, or \$0.02½ per inch. Deducting this from the \$0.03½, gives \$0.01 per square foot net. In the event of rain spoiling a new piece of finish, as frequently happens, or if a slab had to be poured without the finish because of danger of wet weather and the finish laid afterwards, the cost of this finish would be more, and it is usual to allow an extra \$0.01 to cover contingencies on granolithic finish, making the net price of finish \$0.02 per square foot.

Rubbing and cement wash.—The cost of this item depends on how good a job is wanted, and on the quality of work put into the forms. On this job it is estimated at \$0.06 per square foot.

Picked surface and margins, and washing with acid at front entrance.—This is a small job requiring care, and being around

the front door it would be considered the show part of the job, requiring extra treatment. The cost of this was estimated at $20, which is equal to $0.25 per square foot. If there was a large quantity, this could be done for $0.10 per square foot.

Hydrated lime for waterproofing.—Ten per cent is called for in walls and parapet. These structural parts contain 176 cu. yd. of concrete, which would require 293 barrels of cement, equal to 117,200 lb. Ten per cent of this quantity would be 11,720 lb. of hydrated lime, or say 6 tons. The cost of hydrated lime delivered to the job in paper bags would be $15 per ton, and it would not cost any extra to add this waterproofing to the concrete while it is being mixed.

Cinder concrete crickets on the roof.—The estimate of cost would be as follows:

½ barrel of cement	$0.70
½ yd. sand	0.75
1 yd. cinders	0.30
Labor	1.00
Plant	1.25
Total	$4.00 per cubic yard

Cinders can often be obtained free and the cost varies in every locality, running sometimes as high as $1 per cubic yard.

This completes the estimate for the reinforced-concrete work in the building. It does not come within the province of this work to deal with the excavation, windows and doors, roofing, plumbing, painting, and sub-contracting work.

Problem

41. Make a complete estimate for the building designed in Problems 26 to 29 inclusive.

APPENDIX

SECOND REPORT OF JOINT COMMITTEE ON CONCRETE AND REINFORCED CONCRETE

In 1903 and 1904 special committees were appointed by the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers, for the purpose of studying and reporting upon the making and use of concrete, especially reinforced concrete. At a meeting of these special committees, held at Atlantic City, N. J., on June 17, 1904, arrangements were completed for collaborating the work of these several committees through the formation of the Joint Committee on Concrete and Reinforced Concrete. The Joint Committee rendered its first report early in 1909. Critical discussion of this report followed. For the past two years the Joint Committee has been considering changes in the report, and, on Nov. 20, 1912, the revised or second report was adopted by the Committee. The first public presentation of the new report was made to the American Society of Civil Engineers on Jan. 15, 1913.

Following is the report of the Joint Committee as it appeared in the Proceedings of the American Society of Civil Engineers, February, 1913. The greater part of the introduction to the report is omitted.

Since the appearance of the first Progress Report, many experiments have been conducted by some of the technical institutions and by private and corporate interests, and through these and through longer experience in construction by its members and others, the Committee is now able to make some perfecting modifications of its former report and to add some entirely new material. The time, therefore, seems opportune for presenting this second report, bringing the work up to date.

The Committee would point out that while the report deals with every kind of stress to which concrete is subjected, and

includes all ordinary conditions of proportioning and handling, it does not go into all types of construction or all the applications to which concrete and reinforced concrete may be put.

It is not to be assumed that the Committee in presenting this report wishes to imply that further improvements are not possible. A careful reading will disclose many points on which the present deductions are regarded as only tentative; but it has been the aim of the Committee to cover as fully as possible recommendations based on the present state of the art.

This report is what the word implies, and nothing more; it is not a "specification," but may be used as a basis for specifications.

The use of concrete and reinforced concrete involves the exercise of good judgment to a greater degree than for any other building material.

Rules cannot produce or supersede judgement; on the contrary, judgment should control the interpretation and application of rules.

A. Adaptability of Concrete and Reinforced Concrete

The adaptability of concrete and reinforced concrete for engineering structures, or parts thereof, is now so well established that they may be considered the recognized materials of construction. They have proved satisfactory materials, when properly used, for those purposes for which their qualities make them particularly suitable.

1. Uses

Concrete is a material of very low tensile strength, and capable of sustaining but very small tensile deformations without rupture; its value as a structural material depends chiefly on its durability, its fire-resistive qualities, its strength in compression, its relatively low cost, and its adaptability to placing, especially where space is cramped or limited. Its strength increases generally with age.

Concrete is well adapted for structures in which the principal stresses are compressive, such as foundations, dams, retaining and other walls, tunnels, piers, abutments, short columns, and, in many cases, arches. In the design of massive concrete, the tensile strength of the material in resisting principal stresses must generally be neglected.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. In structures resisting lateral forces, it possesses advantages over plain concrete in that it may be designed so as to utilize more fully the strength rather than the weight of the material. Metal reinforcement may also be of value in distributing cracks due to shrinkage and temperature changes.

2. Precautions

Failures of reinforced concrete structures are usually due to any one or a combination of the following causes: defective design, poor material, faulty execution, and premature removal of forms.

The defects in a design may be many and various. The computations and assumptions on which they were based may be faulty and contrary to the established principles of statics and mechanics; the unit stresses used may be excessive, or the details of the design defective.

Articulated concrete structures designed in imitation of steel trusses, may be mentioned as illustrating a questionable use of reinforced concrete, and such structures are not recommended.

Poor material is sometimes used for the concrete, as well as for the reinforcement. The use of poor aggregates, especially sand, which have not been tested, is a common source of defect. Inferior concrete is frequently due also to lack of experience on the part of the contractor and his superintendents, or to the absence of proper supervision.

An unsuitable quality of metal for reinforcement is sometimes prescribed in specifications, for the purpose of reducing the cost. For steel structures, a high grade of material is specified, but the steel used for reinforcing concrete is sometimes made of unsuitable, brittle material.

Faulty execution, careless workmanship, and too early removal of forms may generally be attributed to unintelligent or insufficient supervision.

3. Responsibility and Supervision

The design of reinforced concrete structures should receive at least the same careful consideration as those of steel, and only engineers with sufficient experience and good judgment should be intrusted with such work.

The computations should include all minor details, which are sometimes of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connections between the component parts, so that they cannot be displaced. As the connections between reinforced concrete members are frequently a source of weakness, the design should include a detailed study of such connections, accompanied by computations to prove their strength.

While other engineering structures on the safety of which human lives depend are generally designed by engineers employed by the owner, and the contracts let on the engineer's design and specifications, in accordance with legitimate practice, reinforced concrete structures frequently are designed by contractors or by engineers commercially interested, and the contract let for a lump sum.

The construction of buildings in large cities is regulated by ordinances or building laws, and the work is inspected by municipal authorities. For reinforced concrete work, however, the limited supervision which municipal inspectors are able to give is not sufficient. Therefore, means for more adequate supervision and inspection should be provided.

The execution of the work should not be separated from the design, as intelligent supervision and successful execution can be expected only when both functions are combined. The engineer who prepares the design and specifications, therefore, should have the supervision of the execution of the work.

The Committee recommends the following rules for structures of reinforced concrete for the purpose of fixing the responsibility and providing for adequate supervision during construction:

(a) Before work is commenced, complete plans shall be prepared, accompanied by specifications, stress computations, and descriptions showing the general arrangement and all details. The plans shall show the size, length, dimensions for points of bending, and exact position of all reinforcement, including stirrups, ties, hooping, and splicing. The computations shall

give the loads assumed separately, such as dead and live loads, wind, and impact, if any, and the resulting stresses.

(b) The specifications shall state the qualities of the materials to be used for making the concrete, and the manner in which they are to be proportioned.

(c) The strength which the concrete is expected to attain after a definite period shall be stated in the specifications.

(d) The drawings and specifications shall be signed by the engineer and the contractor.

(e) Plans and specifications for all public structures should be approved by a legally authorized State or City official, and copies of such plans and specifications placed on file in his office.

(f) The approval of plans and specifications by other authorities shall not relieve the engineer or the contractor of responsibility.

(g) Inspection during construction shall be made by competent inspectors employed by and under the supervision of the engineer, and shall cover the following:

1. The materials.
2. The correct construction and erection of the forms and the supports.
3. The sizes, shapes, and arrangement of the reinforcement.
4. The proportioning, mixing, and placing of the concrete.
5. The strength of the concrete, by tests of standard test pieces made on the work.
6. Whether the concrete is sufficiently hardened before the forms and supports are removed.
7. Prevention of injury to any part of the structure by and after the removal of the forms.
8. Comparison of dimensions of all parts of the finished structure with the plans.

(h) Load tests on portions of the finished structure shall be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired. Loading shall be carried to such a point that one and three-quarters times the calculated working stresses in critical parts are reached, and such loads shall cause no injurious permanent deformations. Load tests shall not be made until after 60 days of hardening.

4. Destructive Agencies

(a) *Corrosion of Metal Reinforcement.*—Tests and experience indicate that steel sufficiently embedded in good concrete is well protected against corrosion, no matter whether located above or below water level. It is recommended that such protection be not less than 1 in. in thickness. If the concrete is porous, so as to be readily permeable by water, as when the concrete is laid with a very dry consistency, the metal may corrode on account of the presence of moisture and air.

(b) *Electrolysis.*—The most recent experimental data available on this subject seem to show that while reinforced concrete structures may, under certain conditions, be injured by the flow of electric current in either direction between the reinforcing material and the concrete, such injury is generally to be expected only where voltages are considerably higher than those which usually occur in concrete structures in practice. If the iron be positive, trouble may manifest itself by corrosion of the iron accompanied by cracking of the concrete, and, if the iron be negative, there may be a softening of the concrete near the surface of the iron, resulting in a destruction of the bond. The former, or anode effect, decreases much more rapidly than the voltage, and almost if not quite disappears at voltages that are most likely to be encountered in practice. The cathode effect, on the other hand, takes place even on very low voltages, and is therefore more important from a practical standpoint than that of the anode.

Structures containing salt or calcium chloride, even in very small quantities, are very much more susceptible to the effects of electric currents than normal concrete, both the anode and cathode effects progressing much more rapidly in the presence of chlorine.

There is great weight of evidence to show that normal reinforced concrete structures free from salt are in very little danger under most practical conditions, while non-reinforced concrete structures are practically immune from electrolysis troubles.

The results of experiments now in progress may yield more conclusive information on this subject.

(c) *Sea Water.*—The data available concerning the effect of sea water on concrete or reinforced concrete are limited and inconclusive. Sea walls out of the range of frost action have been standing for many years without apparent injury. In many

harbors where the water is brackish, through rivers discharging into them, serious disintegration has taken place. This has occurred chiefly between low and high tide levels, and is duc, evidently, in part to frost. Chemical action also appears to be indicated by the softening of the mortar. To effect the best resistance to sea water, the concrete must be proportioned, mixed, and placed so as to prevent the penetration of sea water into the mass or through the joints. The cement should be of such chemical composition as will best resist the action of sea water; the aggregates should be carefully selected, graded, and proportioned with the cement so as to secure the maximum possible density; the concrete should be thoroughly mixed; the joints between old and new work should be made water-tight; and the concrete should be kept from exposure to sea water until it is thoroughly hard and impervious.

(d) *Acids.*—Concrete of first-class quality, thoroughly hardened, is affected appreciably only by strong acids which seriously injure other materials. A substance like manure is injurious to green concrete, but after the concrete has hardened thoroughly it resists the action of such acid satisfactorily.

(e) *Oils.*—When concrete is properly made and the surface is carefully finished and hardened, it resists the action of such mineral oils as petroleum and ordinary engine oils. Oils which contain fatty acids produce injurious effects, forming compounds with the lime which result in a disintegration of the concrete in contact with them.

(f) *Alkalies.*—The action of alkalies on concrete is problematical. In the reclamation of arid land, where the soil is heavily charged with alkaline salts, it has been found that concrete, stone, brick, iron, and other materials are injured under certain conditions. It would seem that at the level of the ground-water, in an extremely dry atmosphere, such structures are disintegrated, through the rapid crystallization of the alkaline salts, resulting from the alternate wetting and drying of the surface. Such destructive action can be prevented by the use of a protective coating, and is minimized by securing a dense concrete.

B. Materials

A knowledge of the properties of the materials entering into concrete and reinforced concrete is the first essential. The impor-

tance of the quality of the materials used cannot be overestimated, and not only the cement but also the aggregates should be subject to such definite requirements and tests as will insure concrete of the desired quality.

1. Cement

There are available for construction purposes: Portland, Natural, and Puzzolan or Slag cements. Only Portland cement is suitable for reinforced concrete.

(a) *Portland Cement.*—This is the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials. It has a definite chemical composition varying within comparatively narrow limits.

Portland cement should be used in reinforced concrete construction and any construction that will be subject to shocks or vibrations or stresses other than direct compression.

(b) *Natural Cement.*—This is the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas. Although the limestone must have a certain composition, this composition may vary within much wider limits than in the case of Portland cement. Natural cement does not develop its strength as quickly, nor is it as uniform in composition, as Portland cement.

Natural cement may be used in massive masonry where weight rather than strength is the essential feature.

Where economy is the governing factor, a comparison may be made between the use of natural cement and a leaner mixture of Portland cement that will develop the same strength.

(c) *Puzzolan or Slag Cement.*—This is the finely pulverized product resulting from grinding a mechanical mixture of granulated basic blast furnace slag and hydrated lime.

Puzzolan cement is not nearly as strong, uniform, or reliable as Portland or natural cement, is not used extensively, and never in important work; it should be used only for foundation work underground where it is not exposed to air or running water.

(d) *Specifications.*—The cement should meet the requirements of the Standard Methods of Testing and Specifications for Cement (see Section H), or as may be hereafter amended, the result of the joint labors of Special Committees of the American

Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering Association, and others.

2. Aggregates

Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density[1] or a minimum percentage of voids.

(a) *Fine Aggregate.*—This should consist of sand, crushed stone, or gravel screenings, graded from fine to coarse, and passing when dry a screen having holes ¼ in. in diameter; it is perferable that it be of silicious material, and should be clean, coarse, free from dust, soft particles, vegetable loam, or other deleterious matter; and not more than 6 per cent should pass a sieve having 100 meshes per linear inch. Fine aggregates should always be tested.

Fine aggregate should be of such quality that mortar composed of one part Portland cement and three parts fine aggregate by weight, when made into briquettes, will show a tensile strength at least equal to the strength of 1: 3 mortar of the same consistency made with the same cement and standard Ottawa sand.[2] If the aggregate be of poorer quality, the proportion of cement in the mortar should be increased to secure the desired strength.

If the strength developed by the aggregate in the 1: 3 mortar is less than 70 per cent of the strength of the Ottawa sand mortar, the material should be rejected. To avoid the removal of any coating on the grains, which may affect the strength, bank sands should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture may be determined on a separate sample for correcting weight. From 10 to 40 per cent more water may be required in mixing bank or artificial sands than for standard Ottawa sand to produce the same consistency.

(b) *Coarse Aggregate.*—This should consist of crushed stone or gravel which is retained on a screen having holes ¼ in. in diameter,

[1] A convenient coefficient of density is the ratio of the sum of the volumes of materials contained in a unit volume to the total unit volume.

[2] A natural sand obtained at Ottawa, Ill., passing a screen having 20 meshes and retained on a screen having 30 meshes per lin. in., prepared and furnished by the Ottawa Silica Company, for 2 cents per lb,. f.o.b. cars, Ottawa, Ill., under the direction of the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers.

and graded from the smallest to the largest particles; it should be clean, hard, durable, and free from all deleterious matter. Aggregates containing dust, and soft, flat, or elongated particles, should be excluded from important structures.

The maximum size of the coarse aggregate is governed by the character of the construction.

For reinforced concrete and for small masses of unreinforced concrete, the aggregate must be small enough to produce with the mortar a homogeneous concrete of viscous consistency which will pass readily between and easily surround the reinforcement and fill all parts of the forms.

For concrete in large masses, the size of the coarse aggregate may be increased, as a large aggregate produces a stronger concrete than a fine one, although it should be noted that the danger of separation from the mortar becomes greater as the size of the coarse aggregate increases.

Cinder concrete should not be used for reinforced concrete structures. It may be allowable in mass for very light loads or for fire protection purposes. The cinders used should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal, or ashes.

3. Water

The water used in mixing concrete should be free from oil, acid, alkalies, or organic matter.

4. Metal Reinforcement

The Committee recommends, as a suitable material for reinforcement, steel filling the requirements for structural steel reinforcement of the specifications adopted by the American Railway Engineering Association (Section H).

Where little bending or shaping is required, and also for reinforcement for shrinkage and temperature stresses, material filling the requirements of the specifications adopted by the American Railway Engineering Association for high-carbon steel (Section H) may be used, adopting the same unit stress as hereinafter recommended for structural grade material.

For the reinforcement of slabs, small beams, or minor details, or for reinforcement for shrinkage and temperature stresses, wire

drawn from bars of the grade of rivet steel may be used, with the unit stresses hereinafter recommended.

The reinforcement should be free from excessive rust, scale, or coatings of any character which would tend to reduce or destroy the bond.

C. Preparing and Placing Mortar and Concrete

1. Proportions

The materials to be used in concrete should be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density.

(a) *Unit of Measure.*—The unit of measure should be the cubic foot. A bag of cement, containing 94 lb. net, should be considered the equivalent of 1 cu. ft.

The measurement of the fine and coarse aggregates should be by loose volume.

(b) *Relation of Fine and Coarse Aggregates.*—The fine and coarse aggregate should be used in such relative proportions as will insure maximum density. In unimportant work it is sufficient to do this by individual judgment, using correspondingly higher proportions of cement; for important work these proportions should be carefully determined by density experiments, and the sizing of the fine and coarse aggregates should be uniformly maintained or the proportions changed to meet the varying sizes.

(c) *Relation of Cement and Aggregates.*—For reinforced concrete construction, one part of cement to a total of six parts of fine and coarse aggregates measured separately should generally be used. For columns, richer mixtures are generally preferable, and in massive masonry or rubble concrete, a mixture of 1: 9 or even 1:12 may be used.

These proportions should be determined by the strength of the wearing qualities required in the constrcution at the critical period of its use. Experienced judgment based on individual observation and tests of similar conditions in similar localities is an excellent guide as to the proper proportions for any particular case.

For all important construction, advance tests should be made of concrete, of the materials, proportions, and consistency to be used in the work. These tests should be made under laboratory conditions to obtain uniformity in mixing, proportioning, and storage, and in case the results do not conform to the requirements

of the work, aggregates of a better quality should be chosen, or richer proportions used to obtain the desired results.

2. Mixing

The ingredients of concrete should be thoroughly mixed, and the mixing should continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous. As the maximum density and greatest strength of a given mixture depend largely on thorough and complete mixing, it is essential that the work of mixing should receive special attention and care.

Inasmuch as it is difficult to determine, by visual inspection, whether the concrete is uniformly mixed, especially where limestone or aggregates having the color of cement are used, it is essential that the mixing should occupy a definite period of time. The minimum time will depend on whether the mixing is done by machine or hand.

(a) *Measuring Ingredients.*—Methods of measurement of the proportions of the various ingredients should be used which will secure separate and uniform measurements of cement, fine aggregate, coarse aggregate, and water, at all times.

(b) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass should be used, as a more uniform consistency can be thus obtained. The mixing should continue for a minimum time of at least 1 minute after all the ingredients are assembled in the mixer.

(c) *Hand Mixing.*—When it is necessary to mix by hand, the mixing should be on a water-tight platform, and especial precautions should be taken to turn all the ingredients together at least six times and until they are homogeneous in appearance and color

(d) *Consistency.*—The materials should be mixed wet enough to produce a concrete of such a consistency as will flow into the forms and about the metal reinforcement when used, and which, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

(e) *Retempering.*—Mortar or concrete should not be remixed with water after it has partly set.

3. Placing Concrete

(a) *Methods.*—Concrete, after the completion of the mixing, should be handled rapidly, and in as small masses as is practicable,

from the place of mixing to the place of final deposit, and under no circumstances should concrete be used that has partly set. A slow-setting cement should be used when a long time is likely to occur between mixing and placing.

Concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all the ingredients have settled in their proper places by gravity, and the surplus water has been forced to the surface. Special care should be exercised to prevent the formation of laitance, which hardens very slowly and forms a poor surface on which to deposit fresh concrete. All laitance should be removed.

Before depositing concrete, the reinforcement should be carefully placed in accordance with the plans, and adequate means provided to hold it in its proper position until the concrete has been deposited and compacted; care should be taken to see that the forms are substantial and thoroughly wetted (except in freezing weather) or oiled, and that the space to be occupied by the concrete is free from débris. When the placing of concrete is suspended, all necessary grooves for joining future work should be made before the concrete has had time to set.

When work is resumed, concrete previously placed should be roughened, thoroughly cleansed of foreign material and laitance, thoroughly wetted, and then slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

The faces of concrete exposed to premature drying should be kept wet for a period of at least 7 days.

(b) *Freezing Weather.*—Concrete should not be mixed or deposited at a freezing temperature, unless special precautions are taken to avoid the use of materials covered with ice crystals or containing frost, and to provide means to prevent the concrete from freezing after being placed in position and until it has thoroughly hardened.

As the coarse aggregate forms the greater portion of the concrete, it is particularly important that this material be heated to well above the freezing point.

(c) *Rubble Concrete.*—Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced by the use of clean stones thoroughly embedded in the

concrete as near together as is possible and still entirely surrounded by concrete.

(d) *Under Water.*—In placing concrete under water, it is essential to maintain still water at the place of deposit. The use of tremies, properly designed and operated, is a satisfactory method of placing concrete through water. The concrete should be mixed very wet (more so than is ordinarily permissible) so that it will flow readily through the tremie and into the place with practically a level surface.

The coarse aggregate should be smaller than ordinarily used, and never more than 1 in. in diameter. The use of gravel facilitates mixing and assists the flow of concrete through the tremie. The mouth of the tremie should be buried in the concrete so far that it is at all times entirely sealed and the surrounding water prevented from forcing itself into the tremie; the concrete will then discharge without coming in contact with the water. The tremie should be suspended so that it can be lowered quickly when it is necessary either to choke off or prevent too rapid flow; the lateral flow should preferably be not more than 15 ft.

The flow should be continuous, in order to produce a monolithic mass and prevent the formation of laitance in the interior.

In large structures it may be necessary to divide the mass of concrete into several small compartments or units, filling one at a time. With proper care, it is possible in this manner to obtain as good results under water as in the air.

D. Forms

Forms should be substantial and unyielding, so that the concrete shall conform to the designed dimensions and contours; and they should be tight in order to prevent the leakage of mortar.

The time for removal of forms is one of the most important steps in the erection of a structure of concrete or reinforced concrete. Care should be taken to inspect the concrete and ascertain its hardness before removing the forms.

So many conditions affect the hardening of concrete that the proper time for the removal of the forms should be decided by some competent and responsible person, especially where the atmospheric conditions are unfavorable.

It may be stated in a general way that forms should remain in place longer for reinforced concrete than for plain or massive

concrete, and that forms for floors, beams, and similar horizontal structures should remain in place much longer than for vertical walls.

When the concrete gives a distinctive ring under the blow of a hammer, it is generally an indication that it has hardened sufficiently to permit the removal of the forms with safety. If, however, the temperature is such that there is any possibility that the concrete is frozen, this test is not a safe reliance, as frozen concrete may appear to be very hard.

E. Details of Construction

1. Joints

(a) *Concrete.*—For concrete construction it is desirable to cast the entire structure at one operation, but as this is not always possible, especially in large structures, it is necessary to stop the work at some convenient point. This point should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joints in columns be made flush with the lower side of the girders; that the joints in girders be at a point midway between supports, but should a beam intersect a girder at this point, the joint should be offset a distance equal to twice the width of the beam; that the joints in the members of a floor-system should in general be made at or near the center of the span.

Joints in columns should be perpendicular to the axis of the column, and in girders, beams, and floor slabs perpendicular to the plane of their surfaces.

Girders should never be constructed over freshly formed columns without permitting a period of at least 2 hours to elapse, thus providing for settlement or shrinkage in the columns.

Shrinkage and contraction joints may be necessary in concrete subject to great fluctuations in temperature. The frequency of these joints will depend, first, on the range of temperature to which the concrete will be subjected, and second, on the quantity and position of the reinforcement. These joints should be determined, and provided for in the design. In massive work, such as retaining walls, abutments, etc., built without reinforcement, contraction joints should be provided at intervals of from 25 to 50 ft. and with reinforcement from 50 to 80 ft. (the smaller the height and thickness, the closer the spacing) throughout the length of

the structure. To provide against the structure being thrown out of line by unequal settlement, each section of the wall should be tongued and grooved into the adjoining section. A groove should be formed in the surface of the concrete at vertical joints in walls or abutments.

Shrinkage and contraction joints should be lubricated by either an application of petroleum residuum oil or a similar material, so as to permit a free movement at the joint when the concrete expands or contracts.

The insertion of a sheet of copper or zinc, or even tarred paper, will be found advantageous in securing expansion and contraction at the joint.

(b) *Reinforcement.*—Wherever it is necessary to splice tension reinforcement, the length of lap should be determined on the basis of the safe bond stress, the stress in the bar, and the shearing resistance of the concrete at the point of splice; or a connection should be made between the bars of sufficient strength to carry the stress. Splices at points of maximum stress should be avoided. In columns, bars more than $\frac{3}{4}$ in. in diameter, not subject to tension, should be properly squared and butted in a suitable sleeve; smaller bars may be treated as indicated for tension reinforcement, or the stress may be cared for by embedment in large masses of concrete. At foundations, bearing plates should be provided for supporting the bars, or the bars may be carried into the footing a sufficient distance to transmit the stress of the steel to the concrete by means of the bearing and bond resistance; in no case shall the ends of the bars be permitted merely to rest on concrete.

2. Shrinkage and Temperature Changes

Shrinkage of concrete, due to hardening and contraction from temperature changes, causes cracks the size of which depends on the extent of the mass. The resulting stresses are important in monolithic construction, and should be considered carefully by the designer; they cannot be counteracted successfully, but the effects can be minimized.

Large cracks, produced by quick hardening or wide ranges of temperature, can be broken up to some extent into small cracks by placing reinforcement in the concrete; in long continuous lengths of concrete, it is better to provide shrinkage joints at points

in the structure where they will do little or no harm. Reinforcement is of assistance, and permits longer distances between shrinkage joints than when no reinforcement is used.

Small masses or thin bodies of concrete should not be joined to larger or thicker masses without providing for shrinkage at such points. Fillets similar to those used in metal castings, but of larger dimensions, for gradually reducing from the thicker to the thinner body, are of advantage.

Shrinkage cracks are likely to occur at points where fresh concrete is joined to that which is set, and hence, in placing the concrete, construction joints should be made on horizontal and vertical lines, and, if possible, at points where joints would naturally occur in dimension-stone masonry.

3. Fire-proofing

The actual fire tests of concrete and reinforced concrete have been limited, but experience, together with the results of tests thus far made, indicates that concrete, on account of its low rate of heat conductivity and the fact that it is incombustible, may be used safely for fire-proofing purposes.

The dehydration of concrete probably begins at about 500° F., and is completed at about 900° F., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehydration is very much retarded. The concrete that is actually affected by fire remains in position and affords protection to the concrete beneath it.

The thickness of the protective coating required depends on the probable duration of a fire which is likely to occur in the structure, and should be based on the rate of heat conductivity. The question of the conductivity of concrete is one which requires further study and investigation before a definite rate for different classes of concrete can be fully established. However, for ordinary conditions, it is recommended that the metal in girders and columns be protected by a minimum of 2 in. of concrete; that the metal in beams be protected by a minimum of 1½ in. of concrete; and that the metal in floor slabs be protected by a minimum of 1 in. of concrete.

It is recommended that, in monolithic concrete columns, the

concrete to a depth of $1\frac{1}{2}$ in. be considered as protective covering, and not included in the effective section.

It is recommended that the corners of columns, girders, and beams be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one.

4. Water-proofing

Many expedients have been used to render concrete impervious to water under normal conditions, and also under pressure conditions that exist in reservoirs, dams, and conduits of various kinds. Experience shows, however, that where mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to a rather wet consistency, the resulting mortar or concrete is impervious under moderate pressure.

A concrete of dry consistency is more or less pervious to water, and compounds of various kinds have been mixed with the concrete, or applied as a wash to the surface for the purpose of making it water-tight. Many of these compounds are of but temporary value, and in time lose their power of imparting impermeability to the concrete.

In the case of subways, long retaining walls, and reservoirs, provided the concrete itself is impervious, cracks may be so reduced by horizontal and vertical reinforcement properly proportioned and located, that they are too minute to permit leakage, or are soon closed by infiltration of silt.

Coal-tar preparations, applied either as a mastic or as a coating on felt or cloth fabric, are used for water-proofing, and should be proof against injury by liquids or gases.

For retaining and similar walls in direct contact with the earth, the application of one or two coatings of hot coal-tar pitch to the thoroughly dried surface of concrete is an efficient method of preventing the penetration of moisture from the earth.

5. Surface Finish

Concrete is a material of an individual type, and should not be used in imitation of other structural materials. One of the important problems connected with its use is the character of the finish of exposed surfaces. The finish of the surface should be determined before the concrete is placed, and the work should be

conducted so as to make possible the finish desired. For many forms of construction the natural surface of the concrete is unobjectionable, but frequently the marks of the boards and the flat dead surface are displeasing, making some special treatment desirable. A treatment of the surface, either by scrubbing it while green or by tooling it after it is hard, which removes the film of mortar and brings the aggregates of the concrete into relief is frequently used to remove the form markings, break the monotonous appearance of the surface, and make it more pleasing. The plastering of surfaces, as ordinarily applied, should be avoided, for, even if carefully done, it is likely to peel off under the action of frost or temperature changes.

F. Design

1. Massive Concrete

In the design of massive or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be proportioned so as to avoid tensile stresses, except in slight amounts to resist indirect stresses. This will generally be accomplished, in the case of rectangular shapes, if the line of pressure is kept within the middle third of the section, but, in very large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral forces by reason of their weight, hence the element of weight rather than strength often determines the design. A relatively cheap and weak concrete, therefore, will often be suitable for massive concrete structures.

It is desirable, generally, to provide joints at intervals, to localize the effect of contraction.

Massive concrete is suitable for dams, retaining walls, and piers and short columns in which the ratio of length to least width is relatively small. Under ordinary conditions, this ratio should not exceed six. It is also suitable for arches of moderate span, where the conditions as to foundations are favorable.

2. Reinforced Concrete

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of

concrete and metal is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. The theory of design, therefore, will relate mainly to the analysis of beams and columns.

3. General Assumptions

(a) *Loads.*—The loads or forces to be resisted consist of:

1. *The Dead Load.*—This includes the weight of the structure and fixed loads and forces.
2. *The Live Load.*—This consists of the loads and forces which are variable. The dynamic effect of the live load will often require consideration. Any allowance for the dynamic effect is preferably taken into account by adding the desired amount to the live load or to the live-load stresses. The working stresses hereinafter recommended are intended to apply to the equivalent static stresses thus determined.

   In the case of high buildings, the live load on columns may be reduced in accordance with the usual practice.

(b) *Lengths of Beams and Columns.*—The span length for beams and slabs shall be taken as the distance from center to center of supports, but need not be taken to exceed the clear span plus the depth of beam or slab. Brackets shall not be considered as reducing the clear span in the sense here intended.

The length of columns shall be taken as the maximum unsupported length.

(c) *Internal Stresses.*—As a basis for calculations relating to the strength of structures, the following assumptions are recommended:

1. Calculations will be made with reference to working stresses and safe loads, rather than with reference to ultimate strength and ultimate loads.
2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression, within the usual limits of working stresses, is constant. The distribution of compressive stresses in beams, therefore, is rectilinear.

4. In calculating the moment of resistance of beams, the tensile stresses in the concrete are neglected.
5. Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses, the two materials, therefore, are stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken at 15, except as modified in Section G.
7. Initial stress in the reinforcement, due to contraction or expansion in the concrete, is neglected.

It is recognized that some of the assumptions given herein are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

The deflection of beams is affected by the tensile strength developed throughout the length of the beam. For calculations of deflections, a value of 8 for the ratio of the moduli will give results corresponding approximately with the actual conditions.

4. T-Beams

In beam and slab construction, an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used, extending over the beam and well into the slab.

Where adequate bond and shearing resistance between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall be determined by the following rules:

(a) It shall not exceed one-fourth of the span length of the beam;

(b) Its overhanging width on either side of the web shall not exceed four times the thickness of the slab.

In the design of T-beams acting as continuous beams, due consideration should be given to the compressive stresses at the support.

Beams in which the tee form is used only for the purpose of providing additional compression area of concrete should prefer-

ably have a width of flange not more than three times the width of the stem and a thickness of flange not less than one-third of the depth of the beam. Both in this form and in the beam and slab form, the web stresses and the limitations in placing and spacing the longitudinal reinforcement will probably be controlling factors in design.

5. Floor Slabs

Floor slabs should be designed and reinforced as continuous over the supports. If the length of the slab exceeds one and five-tenth times its width, the entire load should be carried by transverse reinforcement. Square slabs may well be reinforced in both directions.[1]

The continuous flat slab with multiple-way reinforcement is a type of construction used quite extensively, and has recognized advantages for special conditions, as in the case of warehouses with large, open, floor space. At present, a considerable difference of opinion exists among engineers as to the formulas and constants which should be used, but experience and tests are accumulating

[1] The exact distribution of load on square and rectangular slabs, supported on four sides and reinforced in both directions, cannot readily be determined. The following method of calculation is recognized as faulty, but it is offered as a tentative method which will give results on the safe side. The distribution of load is first to be determined by the formula:

$$r=\frac{l^4}{l^4+b^4}$$

in which r = proportion of load carried by the transverse reinforcement, l = length, and b = breadth of slab. For various ratios of $\frac{l}{b}$ the values of r are as follows:

$\frac{l}{b}$	r
1.0	0.50
1.1	0.59
1.2	0.67
1.3	0.75
1.4	0.80
1.5	0.83

Using the values above specified, each set of reinforcement is to be calculated in the same manner as slabs having supports on two sides only, but the total amount of reinforcement thus determined may be reduced 25 per cent., by gradually increasing the rod spacing from the third point to the edge of the slab.

data which it is hoped will in the near future permit the formulation of the principles of design for this form of construction.

The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beam, and its distribution will depend on the relative stiffness of the slab and the supporting beam. The distribution under ordinary conditions of construction may be expected to be that in which the load on the beam varies in accordance with the ordinates of a parabola having its vertex at the middle of the span. For any given design, the probable distribution should be ascertained, and the moments in the beam calculated accordingly.

6. Continuous Beams and Slabs

When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment, and the stresses in concrete recommended in Section G, should not be exceeded. In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

(a) That for floor slabs, the bending moments at center and at support be taken at $\frac{wl^2}{12}$ for both dead and live loads, where w represents the load per linear foot and l the the span length.

(b) That for beams, the bending moment at center and at support for interior spans be taken at $\frac{wl^2}{12}$, and for end spans it be taken at $\frac{wl^2}{10}$ for center and adjoining support, for both dead and live loads.

(c) In the case of beams and slabs continuous for two spans only, the bending moment at the central support should be taken at $\frac{wl^2}{8}$ and near the middle of the span at $\frac{wl^2}{10}$.

(d) At the ends of continuous beams, the amount of negative moment which will be developed will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. There will usually be some restraint, and there is likely to be considerable. Provision should be made for the negative bending moment, but, as its amount will depend on the form of con-

struction, the coefficient cannot be specified here, and must be left to the judgment of the designer.

For spans of unusual length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Even if the center of the span is designed for a greater bending moment than is called for by *a* or *b*, the negative moment at the support should not be taken as less than the values there given.

Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of stress, in accordance with the provisions of Section F, Part 3, c-6. In the case of cantilever and continuous beams, tensile and compressive reinforcement over supports must extend sufficiently beyond the support and beyond the point of inflection to develop the requisite bond strength.

7. Bond Strength, and Spacing of Reinforcement

Adequate bond strength should be provided. The formula hereinafter given for bond stresses in beams is for straight longitudinal bars. In beams in which a portion of the reinforcement is bent up near the end, the bond stress at places in both the straight bars and the bent bars will be considerably greater than for all the bars straight, and the stress at some point may be several times as much as that found by considering the stress to be uniformly distributed along the bar. In restrained and cantilever beams, full tensile stress exists in the reinforcing bars at the point of support, and the bars must be anchored in the support sufficiently to develop this stress.

In case of anchorage of bars, an additional length of bar must be provided beyond that found on the assumption of uniform bond stress, for the reason that, before the bond resistance at the end of the bar can be developed, the bar may have begun to slip at another point, and "running" resistance is less than the resistance before slip begins.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength; but it should be recognized that, even with a deformed bar, initial slip occurs at early loads, and that the ultimate loads obtained in the usual tests for bond resistance may be misleading. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but, as an additional safeguard such anchorage may properly be used in special cases. Anchorage furnished by short

bends at a right angle is less effective than hooks consisting of turns through 180 degrees.

The lateral spacing of parallel bars should not be less than three diameters, from center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than 1 in. The use of more than two layers is to be discouraged, unless the layers are tied together by adequate metal connections, particularly at and near points where bars are bent up or bent down.

8. Diagonal Tension and Shear

When a reinforced concrete beam is subjected to flexural action, diagonal tensile stresses are set up. If, in a beam not having web reinforcement, these stresses exceed the tensile strength of the concrete, failure of the beam will ensue. When web reinforcement, made up of stirrups, or of diagonal bars secured to the longitudinal reinforcement, or of longitudinal reinforcing bars bent up at several points, is used, new conditions prevail; but even in this case, at the beginning of loading, the diagonal tension developed is taken principally by the concrete, the deformations which are developed in the concrete permitting but little stress to be taken by the web reinforcement. When the resistance of the concrete to the diagonal tension is overcome at any point in the depth of the beam, greater stress is at once set up in the web reinforcement.

For homogeneous beams, the analytical treatment of diagonal tension is not very complex—the diagonal tensile stress is a function of the horizontal and vertical shearing stresses and of the horizontal tensile stress at the point considered, and as the intensity of these three stresses varies from the neutral axis to the remotest fiber, the intensity of the diagonal tension will be different at different points in the section, and will change with different proportionate dimensions of length to depth of beam. For the composite structure of reinforced concrete beams, an analysis of the web stresses, and particularly of the diagonal tensile stresses is very complex; and when the variations due to a change from no horizontal tensile stress in the concrete at the remotest fiber to the presence of horizontal tensile stress at some point below the neutral axis are considered, the problem becomes more complex and indefinite. Under these circum-

stances, in designing, recourse is had to the use of the calculated vertical shearing stress as a means of comparing or measuring the diagonal tensile stresses developed, it being understood that the vertical shearing stress is not the numerical equivalent of the diagonal tensile stress, and even that there is not a constant ratio between them. It is here recommended that the maximum vertical shearing stress in a section be used as the means of comparison of the resistance to diagonal tensile stress developed in the concrete in beams not having web reinforcement.

Even after the concrete has reached its limit of resistance to diagonal tension, if the beam has web reinforcement, conditions of beam action will continue to prevail, at least through the compression area, and the web reinforcement will be called on to resist only a part of the web stresses. From experiments with beams, it is concluded that it is safe practice to use only two-thirds of the external vertical shear in making calculations of the stresses that come on stirrups, diagonal web pieces, and bent-up bars, and it is here recommended for calculations in designing that two-thirds of the external vertical shear be taken as producing stresses in web reinforcement.

Experiments bearing on the design of details of web reinforcement are not yet complete enough to allow more than general and tentative recommendations to be made. It is well established, however, that vertical members attached to or looped about horizontal members, inclined members secured to horizontal members in such a way as to insure against slip, and the bending of a part of the longitudinal reinforcement at an angle, will increase the strength of a beam against failure by diagonal tension, and that a well-designed and well-distributed web reinforcement may, under the best conditions, increase the total vertical shear carried to a value as much as three times that obtained when the bars are all horizontal and no web reinforcement is used. Where vertical stirrups are used without being secured to the longitudinal reinforcement, the force transmitted between longitudinal bar and stirrup must not be greater than can be taken through the concrete, and care must be taken to provide for the larger bond stress developed in the longitudinal bars with this construction than exists in the absence of stirrups. Sufficient bond resistance between the concrete and the stirrups or diagonals must be provided. Where the longitudinal bars are bent up, the points of

bending of the several bars should be distributed along a portion of the length of the beam in such a way as to give efficient web reinforcement over the portion of the length of the beam in which it is needed. The higher resistance to diagonal tension failures given by unit frames having the stirrups and bent-up bars securely connected together both longitudinally and laterally is worthy of recognition. It is necessary that a limit be placed on the amount of shear which may be allowed in a beam; for when web reinforcement sufficiently efficient to give very high web resistance is used, at the higher stresses the concrete in the beam becomes checked and cracked in such a way as to endanger its durability as well as its strength.

The section to be taken as the critical section in the calculation of shearing stresses will generally be the one having the maximum vertical shear, though experiments show that the section at which diagonal tension failures occur is not just at a support, even though the shear at the latter point be much greater.

The longitudinal spacing of stirrups or diagonal members, or the distribution of the points of bending of adjacent bent-up bars, should not exceed three-fourths the depth of the beam.

It is important that adequate bond strength or anchorage be provided to develop fully the assumed strength of all web reinforcement.

It should be noted that it is on the tension side of a beam that diagonal tension develops in a critical way, and that the proper connection must always be made between stirrups or other web reinforcement and the longitudinal tension reinforcement, whether the latter is on the lower side of the beam or on its upper side. Where negative moment exists, as is the case near the supports in a continuous beam, web reinforcement, to be effective, must be looped over, or wrapped around, or be connected with, the longitudinal tension reinforcing bars at the top of the beam, in the same way as is necessary at the bottom of the beam at sections where the bending moment is positive and the tension reinforcing bars are at the bottom of the beam.

Inasmuch as the smaller the longitudinal deformations in the horizontal reinforcement are, the less the tendency for the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by arranging and proportioning the horizontal reinforcement so that the unit stresses at points of large shear shall be relatively low.

Where pure shearing stress occurs, or shearing stress combined with but a small amount of tensile stress in the concrete, as when a concentrated load rests on a slab, or other forms of punching shear are produced, or in the case of compression pieces, the element of tension will not need consideration, and the permissible limit of the shearing stress will be higher than the allowable limit when this stress is used as a means of comparing diagonal tensile stress. The working values recommended are given in Section G, Working Stresses.

9. Columns

By columns are meant compression members of which the ratio of unsupported length to least width exceeds about six, and which are provided with reinforcement of one of the forms hereafter described.

It is recommended that the ratio of unsupported length of column to its least width be limited to 15.

The effective area of the column shall be taken as the area within the protective covering, as defined in Section E, Part 3; or, in the case of hooped columns or columns reinforced with structural shapes, it shall be taken as the area within the hooping or structural shapes.

Columns may be reinforced by longitudinal bars, by bands, hoops, or spirals, together with longitudinal bars, or by structural forms which in themselves are sufficiently rigid to act as columns. The general effect of closely spaced hooping is greatly to increase the "toughness" of the column and its ultimate strength, but hooping has little effect on its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of "toughening" are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the possible deformation before ultimate failure.

Composite columns of structural steel and concrete in which the steel forms a column by itself, should be designed with caution. To classify this type as a concrete column reinforced with structural steel is hardly permissible, as the steel will generally take the greater part of the load. When this type of column is used, the concrete should not be relied on to tie the steel units together or to transmit stresses from one unit to another.

The units should be adequately tied together by tie-plates or lattice bars, which, together with other details, such as splices, etc., should be designed in conformity with standard practice for structural steel. The concrete may exert a beneficial effect in restraining the steel from lateral deflection, and also in increasing the carrying capacity of the column. The proportion of load to be carried by the concrete will depend on the form of the column and the method of construction. Generally, for high percentages of steel, the concrete will develop relatively low unit stresses, and caution should be used in placing dependence on the concrete.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

(a) Columns with longitudinal reinforcement only, to the extent of not less than 1 per cent and not more than 4 per cent: the unit stress recommended for axial compression in Section G, Part 3.

(b) Columns with reinforcement of bands, hoops, or spirals, as hereinafter specified: stresses 20 per cent higher than given for *a*, provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.

(c) Columns reinforced with not less than 1 per cent and not more than 4 per cent of longitudinal bars, and with bands, hoops, or spirals, as hereinafter specified: stresses 45 per cent higher than given for *a*, provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.

The foregoing recommendations are based on the following conditions:

In all cases, longitudinal reinforcement is assumed to carry its proportion of stress, in accordance with Part 3. The hoops or bands are not to be counted on directly as adding to the strength of the column.

Bars composing longitudinal reinforcement shall be straight, and shall have sufficient lateral support to be securely held in place until the concrete has set.

Where hooping is used, the total amount of such reinforcement shall be not less than 1 per cent of the volume of the column enclosed. The clear spacing of such hooping shall be not greater than one-sixth of the diameter of the enclosed column, and pref-

erably not greater than one-tenth, and in no case more than $2\frac{1}{2}$ in. Hooping is to be circular, and the ends of bands must be united in such a way as to develop their full strength. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered. The strength of hooped columns depends very much on the ratio of length to diameter of hooped core, and the strength due to hooping decreases rapidly as this ratio increases beyond five. The working stresses recommended are for hooped columns with a length of not more than eight diameters of the hooped core.

Bending stresses due to eccentric loads and lateral forces must be provided for by increasing the section until the maximum stress does not exceed the values above specified; and, where tension is possible in the longitudinal bars, adequate connection between the ends of the bars must be provided to take this tension.

10. Reinforcing for Shrinkage and Temperature Stresses

When areas of concrete too large to expand and contract freely as a whole are exposed to atmospheric conditions, the changes of form due to shrinkage (resulting from hardening) and to action of temperature are such that cracks may occur in the mass, unless precautions are taken to distribute the stresses so as to prevent the cracks altogether, or to render them very small. The distance apart of the cracks, and consequently their size, will be directly proportional to the diameter of the reinforcement and to the tensile strength of the concrete, and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be most effective, therefore, reinforcement (in amount generally not less than one-third of 1 per cent) of a form which will develop a high bond resistance should be placed near the exposed surface and be well distributed. The allowable size and spacing of cracks depends on various considerations, such as the necessity for water-tightness, the importance of appearance of the surface, and the atmospheric changes.

G. Working Stresses

1. General Assumptions

The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be

added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress to be allowed on concrete, we should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class but composed of different materials may have approximately the same degree of safety.

The following recommendations as to allowable stresses are given in the form of percentages of the ultimate strength of the particular concrete which is to be used; this ultimate strength is to be that developed in cylinders 8 in. in diameter and 16 in. long of the consistency described in Section C, Part 2 (*a*), made and stored under laboratory conditions, at an age of 28 days. In the absence of definite knowledge, in advance of construction, as to just what strength may be expected, the Committee submits the following values as those which should be obtained with materials and workmanship in accordance with the recommendations of this report.

Although occasional tests may show higher results than those here given, the Committee recommends that these values should be the maximum used in design.

TABLE OF STRENGTHS OF DIFFERENT MIXTURES OF CONCRETE

(In pounds per square inch)

Aggregate	1 : 1 : 2	1 : 1½ : 3	1 : 2 : 4	1 : 2½ : 5	1 : 3 : 6
Granite, trap rock	3300	2800	2200	1800	1400
Gravel, hard limestone and hard sandstone	3000	2500	2000	1600	1300
Soft limestone and hard sandstone	2200	1800	1500	1200	1000
Cinders	800	700	600	500	400

NOTE.—For variations in the moduli of elasticity see Section G, Part 8.

2. Bearing

When compression is applied to a surface of concrete of at least twice the loaded area, a stress of 32.5 per cent of the compressive strength may be allowed.

3. Axial Compression

For concentric compression on a plain concrete column or pier, the length of which does not exceed 12 diameters, 22.5 per cent of the compressive strength may be allowed.

For other forms of columns the stresses obtained from the ratios given in Section F, Part 9, may govern.

4. Compression in Extreme Fiber

The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses, may be allowed to reach 32.5 per cent of the compressive strength. Adjacent to the support of continuous beams stresses 15 per cent higher may be used.

5. Shear and Diagonal Tension

In calculations on beams in which the maximum shearing stress in a section is used as the means of measuring the resistance to diagonal tension stress, the following allowable values for the maximum vertical shearing stress are recommended:

(a) For beams with horizontal bars only and without web reinforcement calculated by the method given in Section I, Formula (22): 2 per cent of the compressive strength.

(b) For beams thoroughly reinforced with web reinforcement: the value of the shearing stress calculated as for *a* (that is, using the total external vertical shear in Formula (22) for shearing unit stress), must not exceed 6 per cent of the compressive strength. The web reinforcement, exclusive of bent-up bars, in this case, shall be proportioned to resist two-thirds of the external vertical shear in the formulas given in Section I, Formulas (24) or (25).

(c) For beams in which part of the longitudinal reinforcement is used in the form of bent-up bars distributed over a portion of the beam in a way covering the requirements for this type of web reinforcement: the limit of maximum vertical shearing stress (the stress calculated as for *a*), 3 per cent of the compressive strength.

(d) Where punching shear occurs, that is, shearing stress uncombined with compression normal to the shearing surface,

and with all tension normal to the shearing plane provided for by reinforcement: a shearing stress of 6 per cent of the compressive strength may be allowed.

6. Bond

The bond stress between concrete and plain reinforcing bars may be assumed at 4 per cent of the compressive strength, or 2 per cent in the case of drawn wire.

7. Reinforcement

The tensile or compressive strength in steel should not exceed 16,000 lb. per square inch.

In structural steel members, the working stresses adopted by the American Railway Engineering Association are recommended.

8. Modulus of Elasticity

The value of the modulus of elasticity of concrete has a wide range, depending on the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that in computations for the position of the neutral axis and for the resisting moment of beams and for the compression of concrete in columns it be assumed as:

(a) One-fifteenth of that of steel, when the strength of the concrete is taken as 2200 lb. per square inch or less.

(b) One-twelfth of that of steel, when the strength of the concrete is taken as greater than 2200 lb. per square inch, or less than 2900 lb. per square inch, and

(c) One-tenth of that of steel, when the strength of the concrete is taken as greater than 2900 lb. per square inch.

Although not rigorously accurate, these assumptions will give safe results. For the deflection of beams which are free to move longitudinally at the supports, in using formulas for deflection which do not take into account the tensile strength developed in the concrete, a modulus one-eighth of that of steel is recommended.

H. STANDARD SPECIFICATIONS

(a) *Cement*[1]

GENERAL OBSERVATIONS

1. These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2. The Committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

SPECIFIC GRAVITY

3. Specific gravity is useful in detecting adulteration. The results of tests of specific gravity are not necessarily conclusive as an indication of the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

FINENESS

4. The sieves should be kept thoroughly dry.

TIME OF SETTING

5. Great care should be exercised to maintain the test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which the tests are made, a very dry or humid atmosphere, and other irregularities, vitally affect the rate of setting.

CONSTANCY OF VOLUME

6. The test for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

[1] Adopted August 16, 1909, by the American Society for Testing Materials.

7. In making the pats, the greatest care should be exercised to avoid initial strains due to molding or to too rapid drying-out during the first 24 hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

8. The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement, however, may be held for 28 days, and a retest made at the end of that period, using a new sample. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

General Conditions

1. All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, the cement should be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

5. Every facility shall be provided by the contractor, and a period of at least 12 days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages, with the brand and name of manufacturer plainly marked thereon.

7. A bag of cement shall contain 94 lb. of cement net. Each barrel of Portland cement shall contain 4 bags, and each barrel of natural cement shall contain 3 bags of the above net weight.

8. Cement failing to meet the 7-day requirements may be held awaiting the results of the 28-day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society on January 17, 1912, with all subsequent amendments thereto.

10. The acceptance or rejection shall be based on the following requirements:

Natural Cement

11. *Definition.*—This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

FINENESS

12. It shall leave by weight a residue of not more than 10 per cent on the No. 100, and 30 per cent on the No. 200 sieve.

TIME OF SETTING

13. It shall not develop initial set in less than 10 minutes, and shall not develop hard set in less than 30 minutes, or more than 3 hours.

TENSILE STRENGTH

14. The minimum requirements for tensile strength for briquettes 1 sq. in. in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Age.	Strength.
Neat Cement.	
24 hours in moist air	75 lb.
7 days (1 day in moist air, 6 days in water)	150 lb.
28 days (1 day in moist air, 27 days in water)	250 lb.
One Part Cement, Three Parts Standard Ottawa Sand	
7 days (1 day in moist air, 6 days in water)	50 lb.
28 days (1 day in moist air, 27 days in water)	125 lb.

CONSTANCY OF VOLUME

15. Pats of neat cement about 3 in. in diameter, ½ in. thick at the center, tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(a) A pat is then kept in air at normal temperature.

(b) Another is kept in water maintained as near 70° F. as practicable.

16. These pats are observed at intervals for at least 28 days, and, to pass the tests satisfactorily, should remain firm and hard and show no signs of distortion, checking, cracking, or disintegrating.

Portland Cement

17. *Definition.*—This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent has been made subsequent to calcination.

SPECIFIC GRAVITY

18. The specific gravity of cement shall be not less than 3.10. Should the test of cement as received fall below this requirement, a second test may be made on a sample ignited at a low red heat. The loss in weight of the ignited cement shall not exceed 4 per cent.

FINENESS

19. It shall leave by weight a residue of not more than 8 per cent on the No. 100, and not more than 25 per cent on the No. 200 sieve.

TIME OF SETTING

20. It shall not develop initial set in less than 30 minutes; and must develop hard set in not less than 1 hour, nor more than 10 hours.

TENSILE STRENGTH

21. The minimum requirements for tensile strength for briquettes 1 sq. in. in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Age.	Strength.
Neat Cement	
24 hours in moist air	175 lb.
7 days (1 day in moist air, 6 days in water)	500 lb.
28 days (1 day in moist air, 27 days in water)	600 lb.
One Part Cement, Three Parts Standard Ottawa Sand	
7 days (1 day in moist air, 6 days in water)	200 lb.
28 days (1 day in moist air, 27 days in water)	275 lb.

CONSTANCY OF VOLUME

22. Pats of neat cement about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(a) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(b) Another pat is kept in water maintained as near 70° Fahr. as practicable, and observed at intervals for at least 28 days.

(c) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for 5 hours.

23. These pats, to pass the requirements satisfactorily, shall remain firm and hard, and show no signs of distortion, checking, cracking, or disintegrating.

SULPHURIC ACID AND MAGNESIA

24. The cement shall not contain more than 1.75 per cent of anhydrous sulphuric acid (SO_3), nor more than 4 per cent of magnesia (MgO).

(b) *Metal Reinforcement*[1]

6. Steel shall be made by the open-hearth process. Re-rolled material will not be accepted.

7. Plates and shapes used for reinforcement shall be of structural steel only. Bars and wire may be of structural steel or high-carbon steel.

8. The chemical and physical properties shall conform to the following limits:

Elements considered		Structural steel	High-carbon steel
Phosphorus, maximum	Basic	0.04 per cent	0.04 per cent
	Acid	0.06 per cent	0.06 per cent
Sulphur, maximum		0.05 per cent	0.05 per cent
Ultimate tensile strength Pounds, per square inch		Desired 60,000	Desired 88,000
Elong. min. percentage in 8 in., Fig. 1		$\frac{1,500,000}{\text{Ult. tensile strength}}$ [2]	$\frac{1,000,000}{\text{Ult. tensile strength}}$
Character of fracture		Silky	Silky or finely granular.
Cold bends without fracture		180° flat [3]	180° $d=4t$ [4]

[1] Adopted March 16, 1910, by the American Railway Engineering Association.

[2] See Paragraph 15 [3] See Paragraphs 16 and 17. [4] "$d=4t$" signifies "around a pin having a diameter four times the thickness of the specimen."

9. The yield point for bars and wire, as indicated by the drop of the beam, shall be not less than 60 per cent of the ultimate tensile strength.

10. If the ultimate strength varies more than 4000 lb. for structural steel or 6000 lb. for high-carbon steel, a retest shall be made on the same gauge, which, to be acceptable, shall be within 5000 lb. for structural steel, or 8000 lb. for high-carbon steel, of the desired ultimate.

11. Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for, in which case an excess of 25 per cent above the required limits will be allowed.

12. *Plates, Shapes, and Bars.*—Specimens for tensile and bending tests for plates and shapes shall be made by cutting, from the

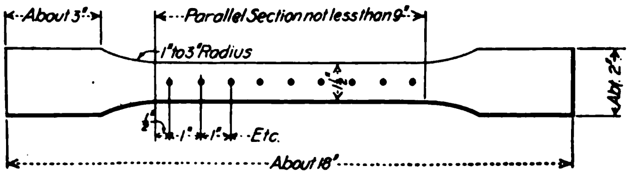

FIG. 1.

finished product, coupons which shall have both faces rolled and both edges milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of $\frac{3}{4}$ in. with enlarged ends.

13. Bars shall be tested in their finished form.

14. At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing $\frac{3}{8}$ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

15. For material less than $\frac{5}{16}$ in. and more than $\frac{3}{4}$ in. in thickness the following modifications will be allowed in the requirements for elongation:

(*a*) For each $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in., a deduction of $2\frac{1}{2}$ will be allowed from the specified percentage.

(*b*) For each $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in., a deduction of 1 will be allowed from the specified percentage.

16. Bending tests may be made by pressure or by blows. Shapes and bars less than 1 in. thick shall bend as called for in Paragraph 8.

17. Test specimens 1 in. thick and greater shall bend cold 180° around a pin, the diameter of which, for structural steel, is twice the thickness of the specimen, and for high-carbon steel, is six times the thickness of the specimen, without fracture on the outside of the bend.

18. Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and have a smooth, uniform, and workmanlike finish.

19. Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled on it, except that bar steel and other small parts may be bundled, with the above marks on an attached metal tag.

20. Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks, or other imperfections, or is found to have injurious defects, will be rejected, and shall be replaced by the manufacturer at his own cost.

21. All reinforcing steel shall be free from excessive rust, loose scale, or other coatings of any character which would reduce or destroy the bond.

I. SUGGESTED FORMULAS FOR REINFORCED CONCRETE CONSTRUCTION

These formulas are based on the assumptions and principles given in the chapter on design.

(a) *Standard Notation*

1. *Rectangular Beams.*

The following notation is recommended:

f_s = tensile unit stress in steel.

f_c = compressive unit stress in concrete.

E_s = modulus of elasticity of steel.

E_c = modulus of elasticity of concrete.

$n = \frac{E_s}{E_c}$.

M = moment of resistance, or bending moment in general.
A = steel area (denoted in this text by a_s).
b = breadth of beam.
d = depth of beam to center of steel.
k = ratio of depth of neutral axis to effective depth d.
z = depth of resultant compression below top.
j = ratio of lever arm of resisting couple to depth d.
$jd = d - z$ = arm of resisting couple.
p = steel ratio (not percentage).

2. *T-Beams.*
b = width of flange.
b' = width of stem.
t = thickness of flange.

3. *Beams Reinforced for Compression.*
A' = area of compressive steel (denoted in this text by a'_s).
p' = steel ratio for compressive steel.
f'_s = compressive unit stress in steel.
C = total compressive stress in concrete.
C' = total compressive stress in steel.
d' = depth to center of compressive steel.
z = depth to resultant of C and C'.

4. *Shear and Bond.*
V = total shear.
v = shearing unit stress.
u = bond stress per unit area of bar.
o = circumference or perimeter of bar.
Σ_o = sum of the perimeters of all bars.

5. *Columns.*
A = total net area.
A_s = area of longitudinal steel.
A_c = area of concrete.
P = total safe load.

(b) *Formulas.*

1. *Rectangular Beams.*

Position of neutral axis,

$$k = \sqrt{2pn + (pn)^2} - pn \tag{1}$$

Arm of resisting couple,

$$j = 1 - \tfrac{1}{3}k \tag{2}$$

(For $f_s = 15{,}000$ to $16{,}000$, and $f_c = 600$ to 650, k may be taken at $\frac{3}{8}$.)
Fiber stresses,

$$f_s = \frac{M}{Ajd} = \frac{M}{pjbd^2}. \tag{3}$$

$$f_c = \frac{2M}{jkbd^2} = \frac{2pf_s}{k} \tag{4}$$

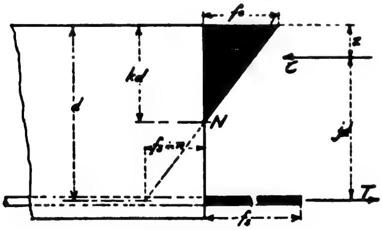

Fig. 2.

Steel ratio, for balanced reinforcement,

$$p = \frac{1}{2} \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c}+1\right)} \tag{5}$$

2. *T-Beams.*
Case I. When the neutral axis lies in the flange.
Use the formulas for rectangular beams.

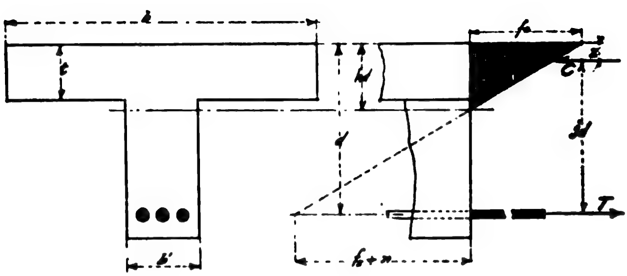

Fig. 3.

Case II. When the neutral axis lies in the stem.
The following formulas neglect the compression in the stem:
Position of netural axis,

$$kd = \frac{2ndA + bt^2}{2nA + 2bt} \tag{6}$$

Position of resultant compression,

$$z=\frac{3kd-2t}{2kd-t}\cdot\frac{t}{3} \tag{7}$$

Arm of resisting couple,

$$jd=d-z \tag{8}$$

Fiber stresses,

$$f_s=\frac{M}{Ajd} \tag{9}$$

$$f_c=\frac{Mkd}{bt\left(kd-\frac{1}{2}t\right)jd}=\frac{f_s}{n}\,\frac{k}{1-k} \tag{10}$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd=\sqrt{\frac{2ndA+(b-b')t^2}{b'}+\left(\frac{nA+(b-b')t}{b'}\right)^2}-\frac{nA+(b-b')t}{b'} \tag{11}$$

Position of resultant compression,

$$z=\frac{\left(kdt^2-\frac{2}{3}t^3\right)b+\left[(kd-t)^2\left(t+\frac{1}{3}(kd-t)\right)\right]b'}{t(2kd-t)b+(kd-t)^2b'} \tag{12}$$

Arm of resisting couple,

$$jd=d-z \tag{13}$$

Fiber stresses,

$$f_s=\frac{M}{Ajd} \tag{14}$$

$$f_c=\frac{2Mkd}{[(2kd-t)bt+(kd-t)^2b']jd} \tag{15}$$

3. *Beams Reinforced for Compression.*

Position of neutral axis,

$$k=\sqrt{2n\left(p+p'\frac{d'}{d}\right)+n^2(p+p')^2}-n(p+p') \qquad (16)$$

Position of resultant compression,

$$z=\frac{\frac{1}{3}k^3d+2p'nd'\left(k-\frac{d'}{d}\right)}{k^2+2p'n\left(k-\frac{d'}{d}\right)} \qquad (17)$$

Arm of resisting couple,

$$jd=d-z \qquad (18)$$

Fiber stresses,

$$f_c=\frac{6M}{bd^2\left[3k-k^2+\frac{6p'n}{k}\left(k-\frac{d'}{d}\right)\left(1-\frac{d'}{d}\right)\right]} \qquad (19)$$

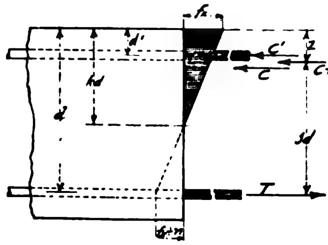

FIG. 4.

$$f_s=\frac{M}{pjbd^2}=nf_c\frac{1-k}{k} \qquad (20)$$

$$f'_s=nf_c\frac{k-\frac{d'}{d}}{k} \qquad (21)$$

4. *Shear, Bond, and Web Reinforcement.*

In the following formulas, Σ_0 refers only to the bars constituting the tension reinforcement at the section in question, and jd is the lever arm of the resisting couple at the section.

For rectangular beams,

$$v=\frac{V}{bjd} \qquad (22)$$

$$u=\frac{V}{jd\,\Sigma_0} \qquad (23)$$

$\left(\text{For approximate results, } j \text{ may be taken at } \frac{7}{8}.\right)$

The stresses in web reinforcement may be estimated by the following formulas:

Vertical web reinforcement,

$$P=\frac{Vs}{jd} \tag{24}$$

Web reinforcement inclined at 45° (not bent-up bars),

$$P=0.7\frac{Vs}{jd} \tag{25}$$

in which P = stress in single reinforcing member, V = amount of total shear assumed as carried by the reinforcement, and s = horizontal spacing of the reinforcing members.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For T-beams,

$$v=\frac{V}{b'jd} \tag{26}$$

$$u=\frac{V}{jd\Sigma_0} \tag{27}$$

(For approximate results, j may be taken at $\frac{7}{8}$.)

5. *Columns.*

Total safe load,

$$P=f_c(A_c+nA_s)=f_cA(1+(n-1)p) \tag{28}$$

Unit stresses,

$$f_c=\frac{P}{A(1+(n-1)p)} \tag{29}$$

$$f_s=nf_c \tag{30}$$

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